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INVESTIGATION OF THE ACTION OF REINFORCED CONCRETE BEAMS

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AN INVESTIGATION OF THE ACTION OF REINFORCED CONCRETE BEAMS

by

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Introductory Note.

Concerning the inception, planning and beginning of the thesis up to the time Mr. Adams began work on the same; and some of the facts which have a bearing as to the accuracy of data obtained.

The thesis of Messrs. Adams and Hahn was not begun by either of them but by Mr. Frank S. Clutz, Ph. D. (Johns Hopkinds) and C.E. (Kansas University.)

Mr. Clutz came to the University of Wisconsin for the second semester, beginning February, 1902, and as a part of his graduate work elected three-fifths "Materials of Construction" besides a thesis in an investigation of steel-concrete beams, the same to be under the direction of Dean J. B. Johnson and under the immediate supervision of the writer.

The objects we had in mind at that time were fourfold. We sought to check the reported marvelous results of M. Considére, Engineer-in-Chief, Ponts et Chaussees, France, to the effect that concrete, when reinforced with iron or steel, will stand from ten to twenty times the elongation up to

rupture that it will when unreinforced. We sought further to see whether or not the cracks of failure of the concrete came, when they did come, according to the ideas presented in Dean J. B. Johnson's "Materials of Construction." We sought further to obtain comparisons as to strength, first crack, etc., with regard to Johnson's corrugated bars, Ransome's twisted bars and plain bars. And, finally, we sought data, of a qualitative kind, to use in the various formulas for steel-concrete design and thus to see which one of them would be of the most general application; in other words, we sought the laws of steel-concrete action or the variables which must be included in a formula for beam action in order to make that formula as rational and broad as possible.

Dean J. B. Johnson had previously written Mr. A. L. Johnson, M.A.S.C.E. of the Expanded Metal Fire-proofing Company, St. Louis, Missouri, for suggestions as to tests, and the reply was a letter which was turned over to the writer of this preface.

This letter suggested cinder and stone concrete beams 2" x 4" x 30" - 24" span, besides other tests. This size of beam gives a ratio of height to length (span) of 1 to 6.

Since M. Considére had used 1 cement to 3 sand for his concrete, and no stone, since our tests involved sawing out slices (as M. Considére did), and since high strength was desired to increase the actual deformations, we decided to

use the same strength concrete. We, therefore, used 1 part of Atlas cement and 3 parts of a local sand, which I located in a pit south of Madison. This sand shows from tests that it could hardly be improved on, especially where strength is desired.

It will be noticed later that we obtained concrete of almost exactly the same strength as Considère did.

As the actual deformations up to the first crack, as well as the strength of beams of the size suggested by Mr. A. L. Johnson, is very small, and as we could procure ready-made bars for larger beams, thereby saving us much labor, we decided to use a beam 3" wide, 6" deep and 50" long with a span of 48", for which a 1/2" bar would be a little more than one percent reinforcement. The ratio of height to length is thus 1 to 8.

This ratio might be criticized as being too large and that we should have used 1 to 10, in order to develop the compressive strength of the beams. In answer to this I will say that Mr. A. L. Johnson's recommendation was 1 to 6 and, furthermore, we did not wish any beams to fail in compression, but, on the contrary, we wished to fully develop the tension side of the beams and to be sure that we had done so. As it is, two of the regular beams failed in compression. Later I designed a reinforcement which, by adding twenty-five percent longitudinal metal, forced a similar beam to fail in

compression. This compression strength was about twenty per-cent greater than we had developed. Then also it must be remembered that we had an extremely strong mixture, especially for compression, as will be seen; and we hardly expected the same to run up to 8000 pounds per square inch, as it did in one case. The data for objects two and four, and possibly three, might have been changed somewhat by the size of our beams, but probably this change would have been slight. On the other hand, for very strong constructions, the ratio of 1 to 8, and even larger ratios, are used very often in practice and thus the data for object four would be even more valuable than otherwise. Let it also be remembered that we made up a second series of beams of size 2-1/2" x 2-1/2" x 26" - 24" span, which gives a ratio of 1 to 9.6, or practically 1 to 10, and that the results on these checked those of the other series. These latter beams were almost exactly the same size as those of M. Considére.

To obtain data at all comparable to M. Considére's, we decided to put the middle 8" of our beams under uniform bending moment. M. Considére had his entire beams under that condition.

To better enable us to watch for the first crack, etc., we placed the beam with its tension side up.

It now became necessary to design a machine to measure the elongation of the extreme fibre. To do this I plan-

ned the apparatus used, called "extensometer." When first built it worked splendidly, giving correct stress-strain curves on tension tests (when the tension attachment was used) and the needle always going around the dial once (as it should) for a half-inch movement between the ends of the beam-attachment when lying flat upon the table. But other students used it in tension tests, some none too carefully, it received one bad fall and it showed increasing frictional resistance, until finally I deemed it wise to check up our results with some other apparatus, especially as our results were getting more erratic. The micrometer apparatus, described later, resulted. It is to be noted here that the yield points in all the beams, as far as load is concerned, almost without exception, were correctly indicated by the extensometer even though it did not record the smaller, initial deformations correctly. Invariably its curve is parallel to that of the micrometer.

The attachment of the extensometer and micrometer bands to the beams may be criticized because of their blunt point nature and lead or other plates advocated as necessary between the concrete and the screw points, but it seems that the results of the final method of taking readings, which give smooth curves and which check other known data, and especially the results of the long-time tests, all answer this criticism themselves. I, for one, believe the final

method adopted to be better than any lead or other plate attachment or to any friction or mirror extensometers which I have ever seen or seen described. Here there is indisputable direct attachment and direct measurement, no slipping or lagging is possible and a disarrangement of the apparatus is immediately detected, as in our long-time test. A series of readings, each one obtained entirely independent of the others and giving a smooth curve, is the highest possible proof of accuracy, and this the method finally adopted gives.

For deflection we first used a scale and silk thread but it was soon seen that a more accurate arrangement was needed, especially for the small beams, and the arrangement described later resulted; in its final form it easily records to 0.001 "correctly and may be read to 0.0001."

Mr. Clutz and myself first obtained some 3/8" square mild steel stuff and made four preliminary test beams. Two oak moulds, thoroughly oiled and shellached, were used for this purpose. These two moulds made all the beams of this size used in the thesis work and while some shrinking was noticed the final beams varied in size from the first ones by an amount which was hardly appreciable.

After making these four beams, we began on the first series of tests planned and made four more. And here is where Mr. Clutz left us to take a position with the American Bridge Company, of Toledo, Ohio. Besides these eight beams,

Mr. Clutz made cross-bending and tension tests on bars, as will be reported later.

It now became necessary to find a new recruit to carry on the work and this was done by Mr. Adams volunteering. Mr. Adams and myself tested the first preliminary lot of beams in the presence of Dean Johnson. These four convinced us that our tests would, without doubt, lead us in the right direction. These four were the only tests Dean Johnson lived to see.

Shortly after this it became necessary to have another recruit as the work in making and testing the beams planned promised to be too great for one man, and so Mr. Hahn volunteered.

That which occurred after Mr. Adams began work will be told by Mr. Adams himself, and that which occurred after Mr. Hahn joined Mr. Adams, will be told by both jointly.

One thing, however, it is fitting that I should mention here and that is the untimely death of Dean Johnson in June, 1902. This came at a time when the work on this thesis was at a standstill, as far as testing goes. However, Messrs. Adams and Hahn worked hard during the summer school session, making long-time beams, and thus accomplished much. Dean Johnson's death took away the director of this series of tests before it had been fairly started and the entire charge of direction and supervision devolved upon me. Due

to the loss of his guidance many difficulties were encountered which undoubtedly would not have arisen had he lived and the actual results from the work done cannot have the effects they would have had.

Rudolph Hartman
Instructor in Testing Laboratory.

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In the original plan of these tests, one series only was laid out. This series was to consist entirely of beams 3" x 6" x 50" reinforced with smooth, corrugated and twisted bars. As the work progressed, other tests suggested themselves and beams with different dimensions were made and tested. All the beams as originally planned and a number of those added later are included in series 1. Series 2 includes an entirely different style of beams and will be taken up in detail later.

Composition and Tests of Mortar.

All beams were made of a mortar composed of one part Atlas Portland cement to three parts of a natural mixture of sand and small gravel obtained from a sandpit near Lake Monona, south of Madison. This mixture was gauged with 8% to 10% of water. The ingredients were proportioned by weight.

Fineness tests were made from each lot of cement and sand used. The cement tests gave the following average,- Percentage rejected by sieve No. 50, 1.3; No. 74, 25.7;

No. 100, 42.3; No. 120, 25.5. No other tests were made on the cement used in these beams. Tests made on the same brand in the laboratory a short time before gave a tensile strength of neat cement briquettes of 750 pounds per square inch in seven days and 950 pounds per square inch in twenty-eight days. "Pat" tests made at the same time show the cement to be sound and without undue expansion during setting. The "pats" were allowed to set under wet clothes for twenty-four hours and were then placed in water at a temperature of 50° to 60° Fahrenheit for one month. The sand and gravel mixture showed in fineness test,- none retained on No. 4 sieve; on No. 8, 22%; on No. 12, 15%; on No. 20, 14%; on No. 30, 18%; on No. 50, 29%. The sand was sharp and fairly clean. The gravel was composed of rounded pebbles.

Method of Making Beams.

In making the beams, the mortar was put in the moulds in thin layers and tamped thoroughly. In nearly all cases the mortar was just wet enough so that, after the tamping, it quaked slightly and water was flushed to the surface or ran out through the joints in the moulds. The reinforcement was designed to be at a distance of two diameters of the rod from the final surface of the beam to the center of the rod. The rod was placed in position after the concrete was brought up to the required height, after which the top layer of concrete was put in place and tamped lightly. The surface was

then finished smooth with trowels.

Setting and Storage of Beams.

All beams were allowed to set for twenty-four hours in the moulds, covered with cloths. After removal from the moulds, with the exception of Group IX of Series 1, all beams were stored in a tank, filled with lake water, until they were tested. The water in the tank was changed once a week, at least. The beams of Group IX were stored in dry sand.

Tests of Mortar.

Briquettes and cubes were made from each batch of concrete mixed for beams. The briquettes were of standard form, minimum section of one sqyare inch. Those briquettes which accompanied beams of Series 1 were tested at seven and twenty-eight days, giving results ranging from 250 pounds per square inch to 300 pounds per square inch, and from 300 pounds per square inch to 450 pounds per square inch, respectively. The briquettes of Series 2 were tested at seven days and three months. Results were about the same as those of Series 1.

The cubes were 3" x 3" x 3" in dimensions and were tested at the same time as the beams which they accompanied. These varied in ultimate strength from 4000 pounds per square inch to 7700 pounds per square inch. Compression curves were

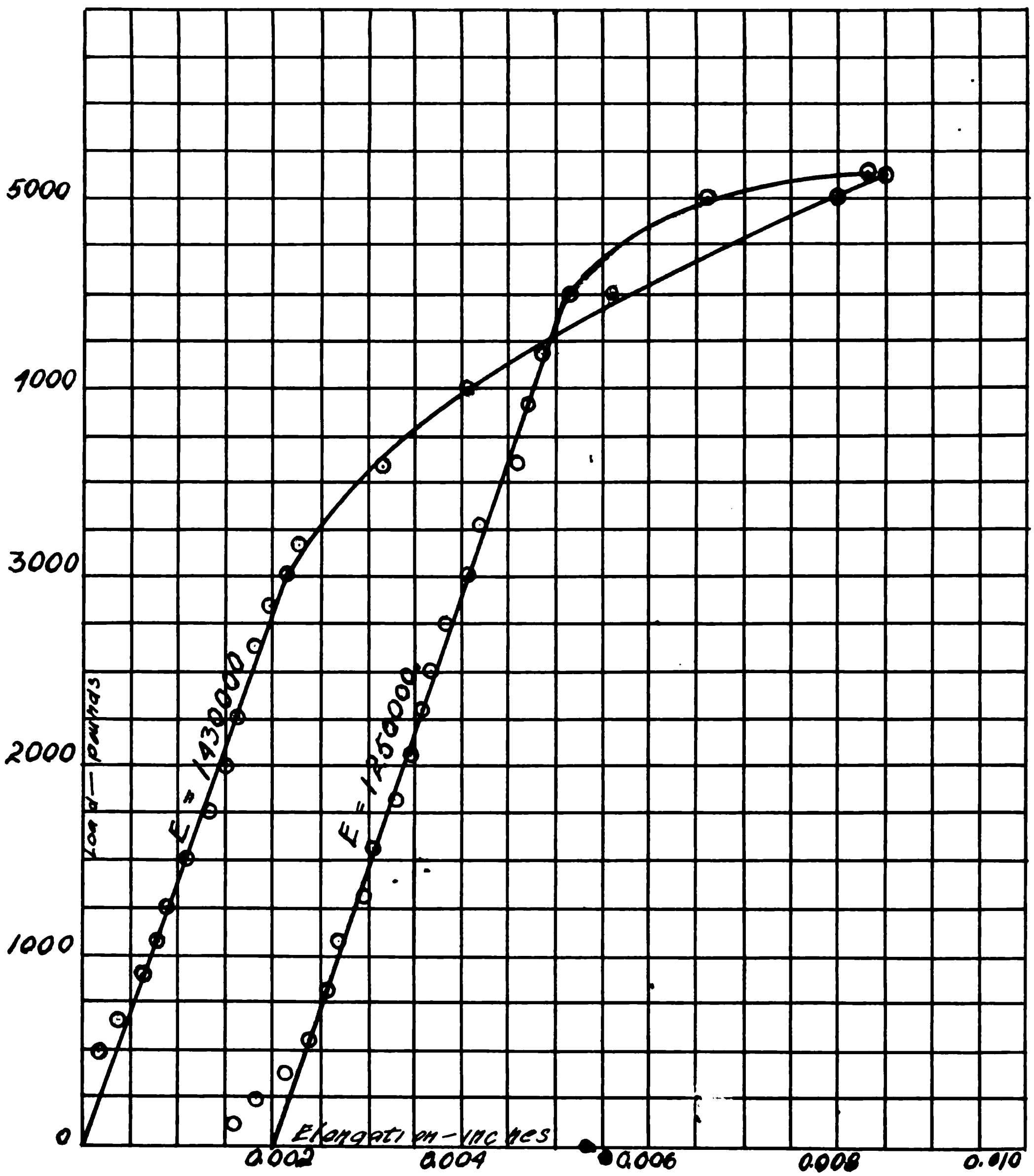
made from a few tests of three months and six months cubes. The compression was measured in all cases with the Olsen compressometer, reading to 0.0001 inch. Measurements were made over a height of three inches. Figure 1 shows two typical curves obtained from these tests. From these curves moduli of elasticity were calculated, which varied from 700,000 to 2,100,000, averaging 1,300,000 for the three months cubes, and varying from 1,000,000 to 2,000,000 for the six months cubes. The average modulus of the latter was about 1,300,000 also.

Objection has been made to the use of such a rich concrete, on the ground that it is not a practical mixture. It is understood perfectly that such a concrete would not be used in actual practice. The object in using this concrete is not to obtain values of constants for practical use in empirical formulas but to get a beam which would be strong enough to take a heavy load before failure, so as to give a fairly wide range of loading over which to take observations.

Description of Testing Apparatus.

The testing machine is shown, in part, in Plates I and II. The weighing apparatus rests on two heavy I beams. The knife edge at the left is fastened to the I beams and remains stationary during the test. The load is applied through the large crosshead at the right. The larger screws

Fig. 1.- Showing Stress-Strain Curves for Concrete Cubes.



on which this crosshead is mounted terminate in a piston in a cylinder beneath the floor. By pumping oil into this cylinder below or above the piston, the crosshead may be raised or depressed.

The beam to be tested is supported on a system of knife edges shown in detail in Figure 2. This arrangement was adopted with two objects in view: First, to have the tension side of the beam uppermost and thus to facilitate the observation of the cracks; second, to obtain a section of uniform bending moment over the middle of the beam. Obviously, we have here a simple beam supported at the ends and bearing two equal concentrated loads symmetrically placed about the center.

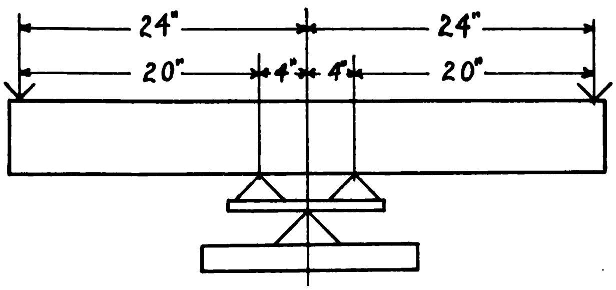


Fig. 2. Arrangement of Knife Edges.

The deflection of beams in Groups I to VII, inclusive, was measured by means of a fine silk thread attached (at one end by a rubber band) to two clamps at the ends of the beam and half way between the top and bottom. A polished steel scale, graduated to 0.01 inch, was fastened to the beam at its center. The movement of the thread over this scale gives the deflection directly.

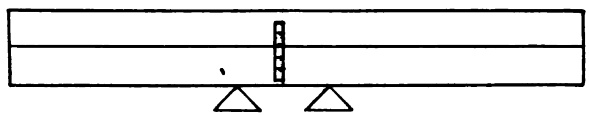
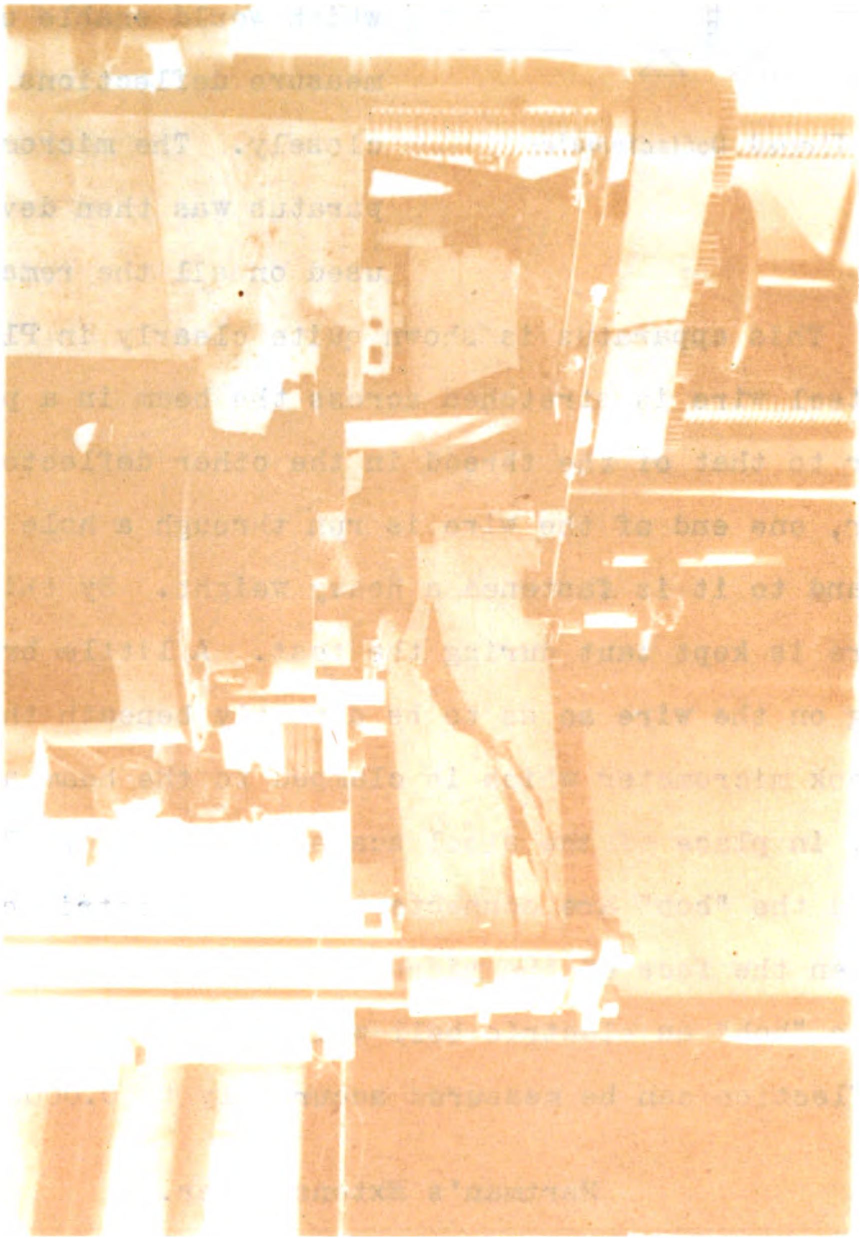


Fig. 3. Thread Deflectometer.

Later it was found advisable to use a deflectometer which would enable us to measure deflections more closely. The micrometer apparatus was then devised and used on all the remaining tests. This apparatus is shown quite clearly in Plate I. A fine steel wire is stretched across the beam in a position similar to that of the thread in the other deflectometer. However, one end of the wire is run through a hole in the clamp and to it is fastened a heavy weight. By this means the wire is kept taut during the test. A little brass "bob" is hung on the wire so as to be directly beneath the screw of a hook micrometer which is clamped to the beam at its center, in place of the steel scale used before. The micrometer and the "bob" are connected into an electric circuit so that when the face of the micrometer screw is in contact with the "bob" an electric bell will ring. By this means, the deflection can be measured accurately to 0.0005 inch.

Hartman's Extensometer.

The elongation was measured over the eight inch section, subject to uniform bending moment, on all beams. The extensometer, designed by Mr. Hartman, was used in all the tests. This instrument is shown in Plate III, in plan. The



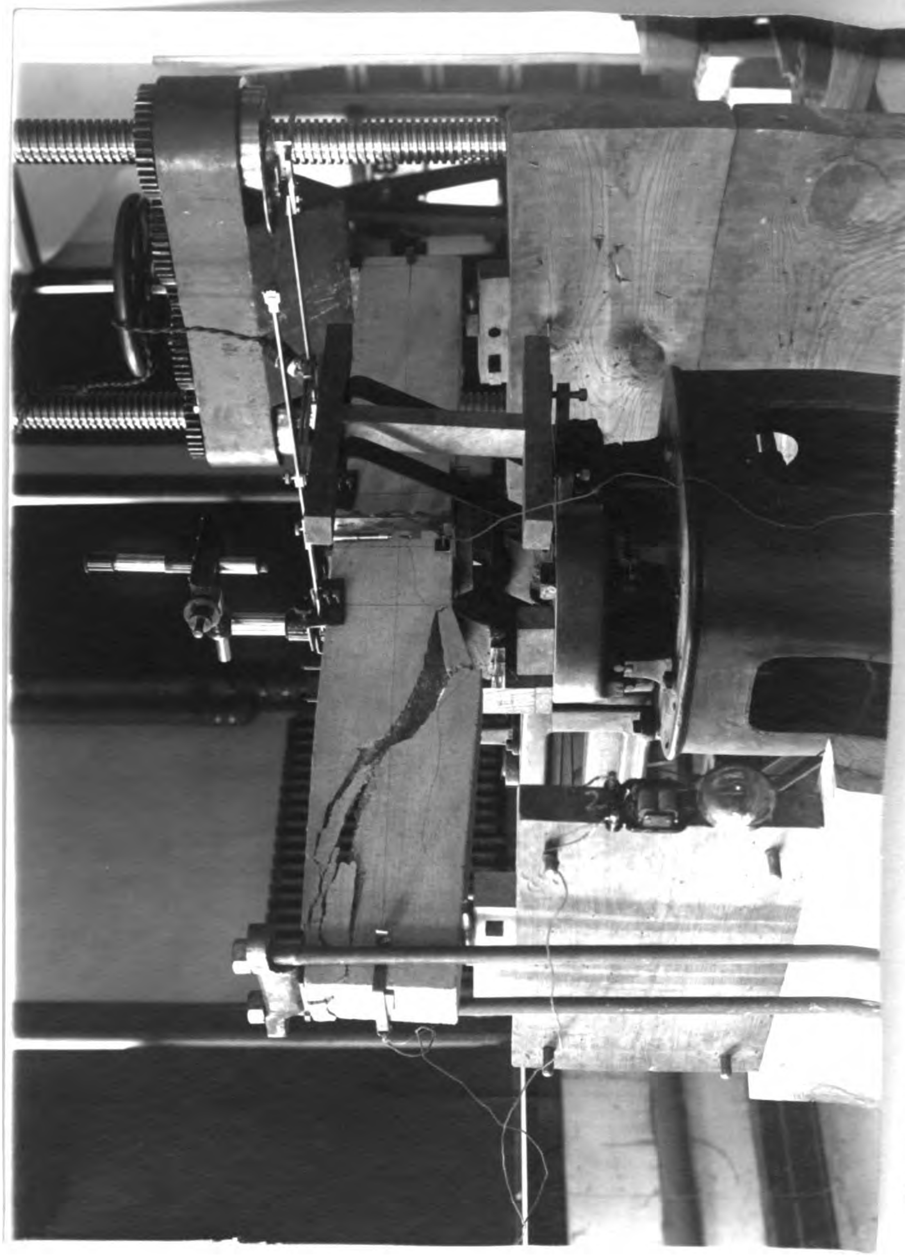


PLATE I

PLATE II.

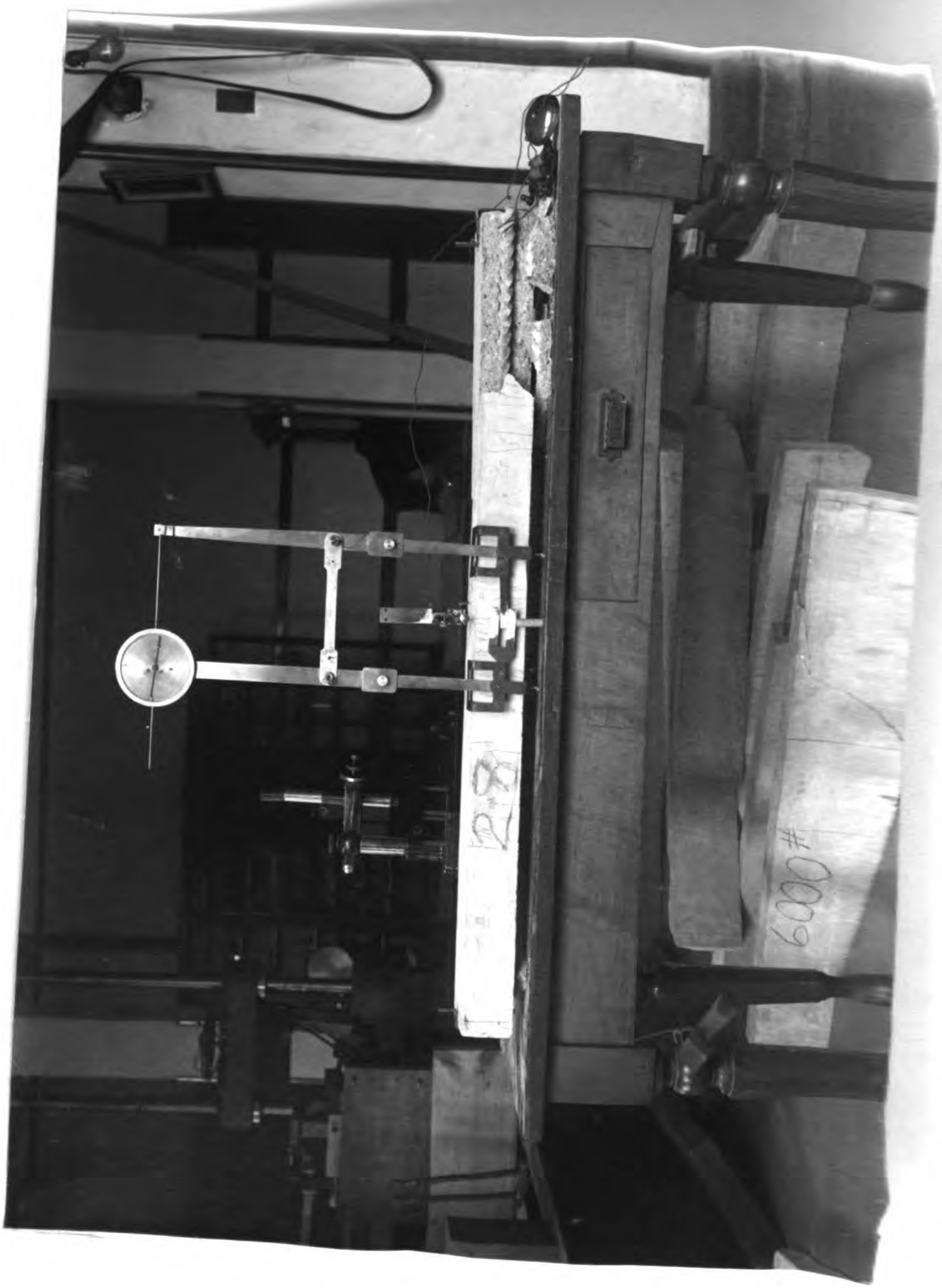


PLATE III.

arms, standing vertically in the photograph, are pivoted at their centers to the cross-bar. Any movement of the ends attached to the beam is reproduced exactly by the other ends. The recording device is that used on the extensometer designed by the late Dean J. B. Johnson, for use in tension tests. On the axis of the needle, behind the dial, is a steel roller one-half inch in circumference. By reason of the friction between this roller and the steel rod extending from the opposite arm, the movement of the rod causes the needle to rotate. A movement of one-half inch at the ends of the arms will cause the needle to rotate through 360 degrees. The dial is graduated to read directly to 0.001 inch and by means of a vernier on the needle a movement of 0.0001 inch may be read. This recording device has been tested many times and its accuracy is conclusively proven.

Filar Micrometer Apparatus.

As the extensometer had not been tested previously to its use on these beams and, also, as it did not seem to be doing its work properly, a device was arranged to work in conjunction with it. This device consisted of two steel plates set on the pivots of the extensometer, over the middle of the beam. The shape and arrangement of these plates are shown in Figure 4 and in Plate III. The plates were polished around the crossmarks. The distance between the

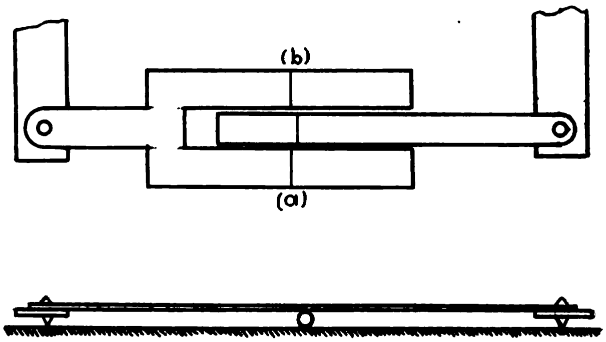


Fig. 4. Plates of Filar Micrometer Apparatus.

marks on the two plates was read with a filar micrometer, after each increment of load was imposed. The filar micrometer used in these tests was obtained from the Department of Physics. It was arranged to read to $\frac{1}{200}$ millimeter, or $\frac{1}{.5008}$ inch. The constant of the instrument when read to millimeters is 0.496 per one turn of the screw, two complete turns being necessary to measure one millimeter. To reduce the actual readings of the disc to ten thousandths of an inch, they should be multiplied by 0.994.

In measuring the elongation with this apparatus, at first, the distance between the marks on the plate was measured only on one side, as at (a) Figure 4. The curves produced by this method were for the most part erratic. A few seemed to be rather more consistent. In general, the results obtained were a little too large. The error was due, in part, to a sidewise motion of the plate caused by the slight rotary movement of the pivots. To avoid this error readings were taken at both (a) and (b), Figure 4. The average of these two movements gives the exact longitudinal movement. This latter method will be referred to as the "double

measurement," as distinguished from the former, or single, measurement method.

Correction of Measurements.

The plates and bars of the extensometer were raised above the surface of the beam one-half inch, by reason of the construction of the extensometer. Therefore the movement measured is not the actual elongation of the extreme fiber of the beam but is a little larger. Making use of the assumption that a plane section of the beam remains plane after bending, which is nearly true, the measured elongation may be reduced to the actual elongation at the surface.

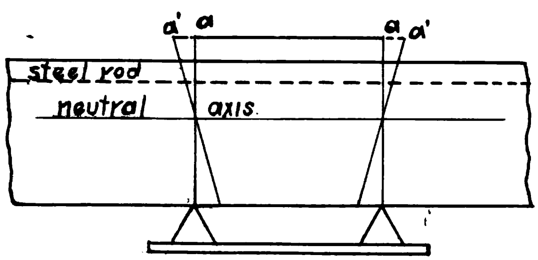


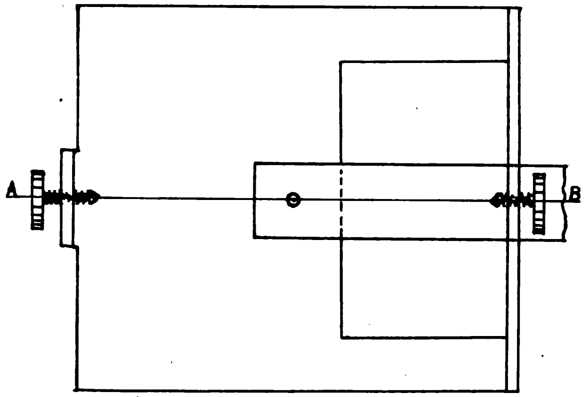
Fig.5.

Figure 5 represents part of the beam. One-half inch above the top surface is the line on which the elongation is measured. When the beam is bent the plane section (a a) under the pivots, originally vertical, will take the position a'a'.

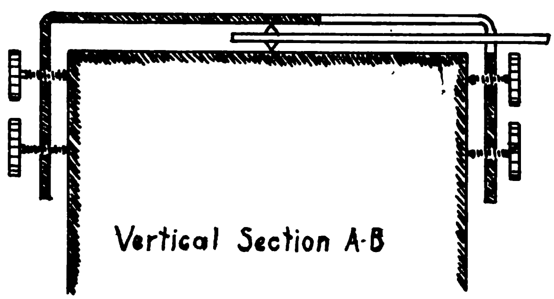
Let m be the measured elongation and n the actual elongation. c is the distance from the neutral axis and d the distance from the neutral axis to the plates.

Then by similar triangles $\frac{m}{n} = \frac{d}{c}$, and $n = \frac{m.c}{d}$. This correction has been applied to all elongations quoted.

The extensometer was fastened to the beam, at first, by means of a wooden clamp. There seemed to be considerable friction between the wood and the pivot so steel clamps were devised to take the place of them. This steel clamp is shown in plan and end-view in Figure 6. It is made of 1/8" steel plate. A shallow conical hole is drilled in the under side of the top to receive the pivot of the extensometer. This socket and the set screws are so placed as to be in a vertical plane when the tops of the clamps are in a horizontal position. The set screws are of brass and blunt pointed. They are set so as to be as near the top edge of the beam as possible and yet have a firm bearing. On account of imperfection in the manufacture of the clamp, the top screw on one side was a little higher than the opposite one. In several cases this brought it so high that it could not be used and the clamp was held in place only by three screws. This clamp has given good satisfaction.



View - Looking Upward.



Vertical Section A-B

Fig. 6.

Series Number 1.

Objects.

The original objects for which the tests included in this series were made have been given in the introduction. In addition to these it was found desirable to make tests on a few unreinforced beams, in order to find the elongation in concrete when subjected to flexural stress. Group I of beams reinforced with 3/8" smooth rods, while originally intended only for practice in running the test, and Groups X, XI and XII are offered as giving a comparison with the other groups of this series of the effects due to various proportions of metal. Group IX, the beams of which were packed in dry sand, was designed to determine what difference, if any, there was between the beams set in water and those set in air.

Table 1 gives in detail the beams in each group, their dimensions, reinforcement and results of the tests, including tensile and compression strengths of the respective mortars.

The beams of this series, where reinforced, have, each, a steel bar placed two diameters beneath the tension surface.

Description of Reinforcement.

Plate IV shows the various bars used and their manner

Table No. 1.— Data of Beams in Series No. 1.

No.	Load 1st Seen Water Mk	Yield Point Load	Yield Point Defl'n	Elongation Measured	Elongation Computed	Load 1st Seen Crack	Load at Failure	Tension Tests 7 days	Tension Tests 28 days	Compression Tests
	pounds	pounds	inches	inches per inch						
Group I.		Beams 3" × 6" × 50",			1 month old		3/8" smooth rods.			
1		1500	0.021	0.00006			2600			
2		1250	0.008	0.00017			3000			
3		1350	0.021	0.00006			2700			
4		1250	0.026	0.00006			2000			
Group II.		Beams 3" × 6" × 50",			1 month old		1/2" smooth rods.			
35	1400	1200	0.010		0.00011	1800	4000	330	419	4350
36	1400	1600	0.012		0.00025	2250	5250	285	370	4950
37	1600	1400	0.011		0.00012	2200	4100	280	360	4390
38	1400	1600	0.021	0.00015	0.00035	1800	4500	310	402	3960
39	1600	1600	0.018	0.00010	0.00020	2200	4400	290	370	4300
40	1300	1700	0.026	0.00015	0.00030	2200	4500	240	320	3960
Group III.		Beams 3 × 6 × 50",			3 months old.		1/2" smooth rods.			
18	2100	1600	0.022		0.00026	2300	2800	262	333	4530
19	1400	1700	0.012	0.00006	0.00014	2300	4200	244	340	4630
20	1400	1700	0.017	0.00006	0.00020	2400	6000	300	445	6600
21	2000	1800	0.014		0.00018	2800	5600	400	515	4980
22	1600	1600	0.010		0.00011	3000	6500	450	527	2070
Group IV.		Beams 3" × 6" × 50".			3 months old.		Twisted rods.			
30	2000	1800	0.010	0.00001	0.00011	2200	5500	346	376	4975
31	2000	1600	0.016	0.00008	0.00020	2200	5000	340	415	4300
32		1600	0.015	0.00010	0.00020	2400	4600	318	410	4840
33		1800	0.022	0.00015	0.00026	2600	4600	262	383	5010
34	1800	2000	0.023		0.00028	2600	5800	274	386	4625
Group V.		Beams 3" × 6" × 50".			3 months old.		corrugated rods.			
11	1500	1500	0.013	0.00006	0.00015	1900	4400	300	442	4610
12	1800	1800	0.017	0.00010	0.00020	2100	4600	305	370	
13	1200	1900	0.025	0.00011	0.00030	2100	4650	268	344	4320

Table No 1. – Data of Beams in Series No 1. cont'd.

NO.	LOAD 1st Seen Water Mk	YIELD POINT LOAD	YIELD POINT DEFL'N	YIELD POINT Elogation Measured	YIELD POINT Elogation Computed	LOAD 1st Seen Crack	LOAD at Failure	TENSION TESTS 7 days	TENSION TESTS 28 days	COMPRESSION TESTS
Group V. cont'd										
23	2000	2000	0.016		0.00020	2200	5500	430	520	5650
24	1800	2000	0.025	0.00010	0.00030	2250	5500	350	520	5910
Group VI Beams 3"×6"×50"; 6 months old; ½" smooth rods.										
14	1800	1800	0.016	0.00005	0.00020	2450	5300	290	340	3720
15	1800	1800	0.022	0.00020	0.00026	2200	4900	210	250	4050
16	2000	2000	0.025	0.00010	0.00030	2800	5600	300	350	5450
17	1800	1800	0.022	0.00005	0.00026	2200	4600	225	400	4580
Group VII. Beams 3"×6"×50"; 6 months old; corrugated rods.										
5	1200	1200	0.017	0.00010	0.00020	1400	6200	235	380	4400
6	1600	1700	0.017	0.00005	0.00020	2200	5000	235	380	3900
7	2000	1500	0.013		0.00015	2200	6000	225	340	4750
8	1400	2000	0.022	0.00020	0.00026	2250	5400	226	380	3490
10	1600	1800	0.019		0.00024	2200	4200	360	420	5880
Group VIII. Beams 3"×6"×50"; 6 months old; twisted rods.										
25	2400	2400	0.016	0.00008 e 0.00011 m	0.00030	2800	6000	400	540	
26	1800	2000	0.027	0.00005 e 0.00023 m	0.00030	2000	2200	350	450	5930
27		2000	0.020		0.00024	2000	2640	325	400	
28	1800	2400	0.020	0.00025 m	0.00024	2640	5660	420	500	7800
29	2150	1600	0.014		0.00016	2000		337	420	5100
Group IX. Beams 3"×6"×50"; 3 months old; corrugated rods; stored dry.										
57		900	0.010	0.00023 m	0.00011	1500	6420	255	414	
58		1100	0.013	0.00012 m	0.00015	1500		255	414	
61		1000	0.014	0.00020 m	0.00016	1800	5200	250	452	5100
62		1000	0.015	0.00020 e 0.00030 m	0.00020	1800	4600	250	452	5100
Group X. Beams 4"×6"×50" 3 months old; corrugated rods.										
45	1600	2200	0.020	0.00022 e 0.00038 m	0.00024		6300	280	495	5500
46	1600	2000	0.020	0.00011 e 0.00026 m	0.00024		6500	280	495	6130

Note: Groups VIII–X, Elongations,– e – extensometer, m – micrometer.

Table Nº 1. Data of Beams in Series Nº 1. cont'd.

NO.	LOAD 1st Seen Water Mk.	YIELD POINT LOAD	YIELD POINT DEFL'N	YIELD POINT Elongation Measured	YIELD POINT Elongation Computed	LOAD 1st Seen Crack	LOAD at Failure	TENSION TESTS 7 days	TENSION TESTS 28 days	COMPRESSION TESTS
Group XI Beams 4"×6"×50"; 3 months old; twisted rods.										
48	2200	2000	0.015	0.00015m	0.00020		5920	215	397	4600
49	2000	2000	0.016	0.00020m	0.00020		6000	215	397	4600
Group XII. Beams 4"×6"×50"; 3 months old; ½" smooth rods.										
51	2600	1800	0.013		0.00016		6000	198	440	5100
52	2400	2000	0.015	0.00025m	0.00020		6600	198	440	5100
Group XIII. A. Beams 3"×6"×50"; 3 months old; unreinforced.										
41	1000	1200	0.014	0.00026m	0.00018		1200	300	340	6600
42		1250	0.012	0.00025m	0.00015		1250	370	380	5250
43	1000	1200	0.013	0.00036e 0.00036m	0.00016		1200	290	480	4300
44		1150	0.013	0.00014e 0.00026m	0.00016		1150	280	435	5450
Group XIII. B. Beams 3"×6"×26".										
41		3100	0.0040	0.00023m	0.00016		3100			
42		3340	0.0045	0.00030m	0.00018		3340			
43		3250	0.0050	0.00045m	0.00020		3490			
44		3440	0.0050	0.00030m	0.00020		3440			

Measured elongations:– m = micrometer, e = extensometer.

of failure in tension. Numbers 1 and 2 are specimens of Ransome's twisted bar. This bar is a 1/2" square bar twisted cold. The elastic limit is about 60,000 pounds per squate inch in the case of those used in these tests. Numbers 4 and 6, 16 and 17, are the 1/2" and 3/8" smooth bars having elastic limits of 60,000 pounds per square inch and 48,000 pounds per square inch respectively. Numbers 8, 10, 12, and 14 are specimens of Mr. A. L. Johnson's old style corrugated bar. These bars are rolled from 1/2" square stock and have a minimum section of 0.19 pounds per square inch. Attention is called to the angle of the transverse corrugations with respect to the axis of the bar. In Numbers 8, 10 and 12, the corrugations are perpendicular to the axis. In Number 14 they are oblique to the axis. An examination of one of these bars shows that on two sides the corrugations are perpendicular to the axis, as in Numbers 8, 10 and 12. On the other two sides the corrugations are oblique, as in Number 14. Numbers 8 and 10 are specimens of the rods used in our beams. Numbers 12 and 14 are specimens of another lot, much inferior in strength and elastic limit, and more brittle. The elastic limit of the rods used in these beams was about 60,000 pounds per square inch.* Numbers 3, 5, 7, 9, 11, 13 and 15 are the steel wires used in the beams of Series 2. These vary by thirty-seconds of an inch from 1/16" to 1/4" in

*The tension tests of these bars were made by Messrs. Hartman and Clut-

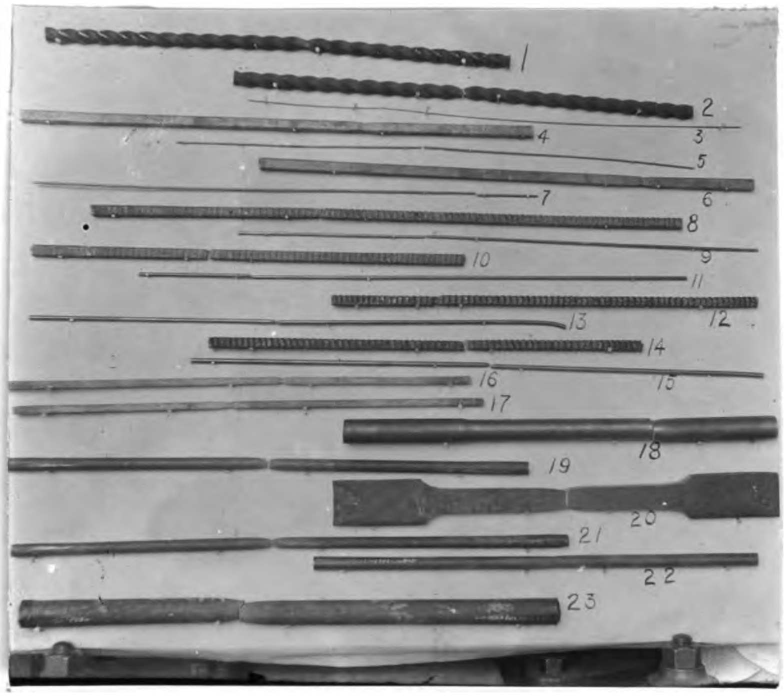


PLATE IV

diameter. They are of Bessemer with elastic limits ranging from 65,000 per square inch to 90,000 pounds per square inch. Numbers 18 to 23 inclusive are normal tension specimens of cast-iron mild steel, boiler plate steel, Swedish wrought iron, tool steel and mild steel, respectively. A comparison of the fractures of these specimens with the fractures of specimens Numbers 1 to 17 will show, to some extent, the quality of steel of which the latter are composed.

Unreinforced Beams.

Group XIII consists of four unreinforced beams. Elongations were measured with the micrometer apparatus with single measurement, and with the extensometer. Deflection was measured with the electrical contact apparatus. The beams were tested over a span of 48 inches. Loads at failure ranged from 1150 pounds to 1200 pounds; deflection from o.012 inches to 0.014 inches; measured elongations are 0.00026 inches, 0.00025 inches, 0.00026 inches and, in case of Number 43, 0.00036 inches per inch respectively. These beams failed suddenly near the middle in each case.

After finishing these four tests, the longer piece of each broken beam was tested on a span of 24 inches. As before, the deflection curves were straight lines, or practically so. The elongations were 0.00023, 0.00030, 0.00030, and, as before, in case of beam No. 43, 0.00045 inches per

inch. The deflections varied from 0.0045 inch to 0.005 inch. The elongations, as computed from the deflection of the short span beams, are 0.00020, 0.00023, 0.00024 and 0.00024, respectively. As may be seen, the elongation of these shorter beams shows a slight increase over that of the long beams. This may be due to the fact that the beams were one month older when tested on the short span.

Beam Number 43.

Beam No. 43 showed a peculiar elongation curve in each test. A very distinct bend may be seen at about 80 percent of the load at failure in both curves. At the same time the total elongation was much larger than that obtained on the other three beams. A possible explanation is that there was a slight local failure of the middle fibers on the top surface of the beam, which was not large enough to cause rupture clear across. Elongation curves of these six tests are given in Figure 12, page 48.

Water-Marks.

During the tests of the first few reinforced beams a most interesting phenomenon was noticed. This was the appearance of the so-called water-marks on the top and sides. In this connection it will be recalled that the upper sides of the beam in these tests was in tension. If the surface of the wet beam is allowed to dry off before testing, after

a certain load has been imposed, wet spots appear on the tension face. These spots grow larger as the load is increased and extend gradually across the top and down the sides. Almost invariably a crack opens up in the space covered by the wet spot or water-mark. This action was closely observed on nearly all beams of this series and Series 2. The exceptions were Numbers 57, 58, 61 and 62 of Group IX and such of the wet beams as were not allowed to dry off before testing, or in two or three cases had become too dry.

Explanation of Water-Marks.

Our theory, with regard to these water-marks, was that when the concrete cracked slightly the water in the pores underneath the surface was forced to the surface through these very narrow cracks by capillary action, and spread out over the surface. In order to obtain evidence for or against this theory, we made special tests on beam No. 29, Group VIII.

Tests of Beam Number 29.

The beam was loaded and curves taken in the usual manner until two water-marks appeared. These were developed until they extended across the top of the beam. No crack was visible to the naked eye but with the aid of the micrometer microscope, a crack was traced for about a quarter of an inch in the first water-mark. No crack could be made out

in the second large water-mark, even with the aid of the microscope. With the addition of the last load, three more small water-marks appeared, two on the top surface and one on the edge of the beam. These last water-marks were very small. The positions of the water-marks are indicated in Plate V, by the black lines across the beams and the small ellipses.

A section was detached from this side of the beam, between the steel rod and the surface, by means of stone-saw. The saw used for this purpose consisted of an endless iron wire which ran between two grooved pulleys. Emery powder was fed in between the concrete and the wire, to cut away the concrete. When the saw reached the first large water-mark, shown on the right in Plate V, the slab of concrete fell out. In sawing toward the left end of the beam nothing happened as the saw passed the little water-mark, but as the saw reached the other large water-mark this slab fell out. Two other slabs were taken out which had on them no signs of cracks or water-marks. The slabs as taken out are shown in Plate VI. The first cut was made between 2 and 3. The fact that the slabs dropped out, breaking apart at the larger water-marks, is conclusive proof that there were cracks present at these points.

These four slabs were later tested in cross-bending on a five inch span, with center load. The results obtained

PLATE V

PLATE VI

are shown in Table No. 2. On Numbers 1 and 2 were the two small water-marks mentioned above, 1/4" and 2" from the center of span, respectively. Number 1 failed through the water-mark. Number 2 failed at the center but a crack had opened through the water-mark. Numbers 3 and 4 both failed through the middle. The failure of Numbers 1 and 2 is also good evidence that there were cracks through the water-marks.

Table No. 2: - Tests of Concrete Slabs from Beam 29.

Number	Dimensions		Load at Failure	Modulus of Rupture
	height inches	breadth inches		
1	0.64	3.0	120	734
2	0.72	3.0	150	725
3	0.58	3.0	115	870
4	0.67	3.0	160	890

Finely divided Cracks.

The strength of these small slabs would show that there was no disintegration of the concrete such as would be caused by the occurrence of finely divided small cracks, as described in Johnson's Materials of Construction, page 72c, edition of 1901. It would seem that this evidence is enough, in connection with that set forth by M. Considére* to disprove the above theory of the occurrence of fine cracks.

*Influence des Armatures Metallique sur les Propriétes des Mortiérs et Bétons. Le Génie Civil, Feb. 4, 1899.

Water-Marks on Unreinforced Beams.

In case of the unreinforced beams, Group XIII, no water-marks appeared on Numbers 41, 42 and 44. On No. 43, which showed an unusually large elongation, small water-marks appeared in the eight inch section.

Time of Appearance of Water-Mark.

Quite a wide variation in load at appearance of the water-mark, as recorded, is noticeable. This is, without doubt, because of the fact that their appearance is, in a way, a function of the wetness of the beam. In some cases tests were started on beams before their surfaces were wholly dry. The surface dried unevenly and wet spots sometimes remained which resembled closely the real water-marks, and may have been recorded as such. Again, when a beam had been left out of water for an hour or more before testing, the surface and the body of the beam directly under the surface was dried out a little more than usual and the watermarks did not show up till after their usual time for appearance. This is explained by the fact that the crack occurs first at the surface and then extends gradually down into the body of the beam. At the same time a slight variation in the strength and modulus of elasticity of the concrete, in different beams, would affect both the load and deflection of the beams at the point of frist crack. With a heterogeneous

material like concrete it would be almost impossible to keep the conditions so uniform that the strength and modulus would be exactly the same in several specimens.

Yield Point.

An examination of the load-deflection curves in Figures 19 and 20 show them to consist of a line practically straight up to a point corresponding to a load of 1600 pounds to 2000 pounds, and to a deflection of 0.016 inch to 0.028 inch. With further increase of load the curve bends rapidly toward the deflection axis. This point of inflection will be called the "yield point."

Coincidence of Yield Point, Water-Mark and Crack.

From Table No. 1 it may be seen that, in general, the loads at yield point and first noticed water-mark correspond almost exactly. Allowing for the variations explained above, the coincidence seems to warrant the statement that the yield point is actually the point at which the first crack occurs in the concrete. These tests are corroborated by nearly all the tests made in the second series, to be described in another part of this paper.

It must be remembered that this proposition is considered as proved only with regard to beams stored in water up to the time of testing. All of the four beams which were

stored in sand give precisely similar curves and cracks were found comparatively near to the yield point. Naturally, there were no water-marks on these beams and without them it is practically impossible to find the cracks until they are quite well developed.

Professor W. K. Hatt, of Purdue University, writing of "Tests of Concrete-Steel Beams,"* gives a series of load-deflection curves of beams stored in dry sand, tested with center load and with the tension side down. These curves show the yield point distinctly. Figure 7 shows one of his load-deflection diagrams, as given in the Engineering Record, Vol. 45, p. 605. Professor Hatt calls this yield point, "point A." Further than computing values for the extreme fiber stress, stress in the steel and position of the neutral axis at this point, he has nothing to do with it.

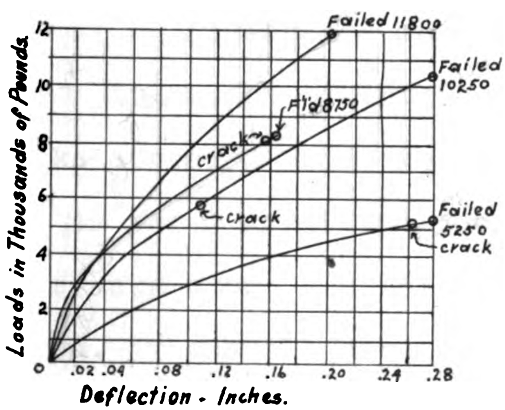


Fig.7

At about 80 percent of the final breaking load of his beams, Professor Hatt records the appearance of the first crack. Undoubtedly the crack was recorded as soon as it was

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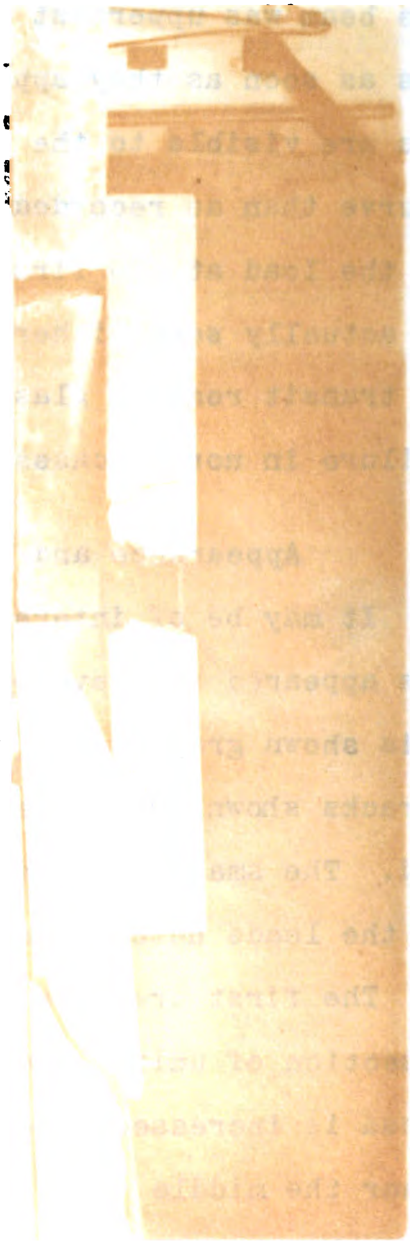
*Engineering News, Vol. 47, p. 170.
Engineering Record, Vol. 46, pp.
Railroad Gazette, Vol. 34, p. 773.

seen, but it was probably not seen until pretty well developed on the side of the beam. In our tests, the tension side of the beam was uppermost and it was possible to see the cracks as soon as they appeared. As a matter of fact the cracks are visible to the naked eye at a much lower point on the curve than as recorded by Professor Hatt. Table No. 1 shows the load at the first noticed crack, that is, the first crack actually seen either with the naked eye, or with a small transit reading glass, at about 35% to 40% of the load at failure in normal cases.

Appearance and Development of Cracks.

It may be of interest to note the manner in which the cracks appeared and developed on the surface of the beam. This is shown graphically on the sketches, Figure 8. The cracks shown are lettered in the order in which they appeared. The small circles indicate the points to which the cracks had progressed at the loads noted alongside.

The first cracks almost invariably occur in the eight inch section of uniform bending moment. (a a, Figure 9); as the load is increased other cracks appear, usually one or two near the middle (b b, Figure 9), and several outside the eight inch section. It was noticeable that the cracks in the six-months beams were much more numerous than on the one-month and three-months beams. This was true both inside and



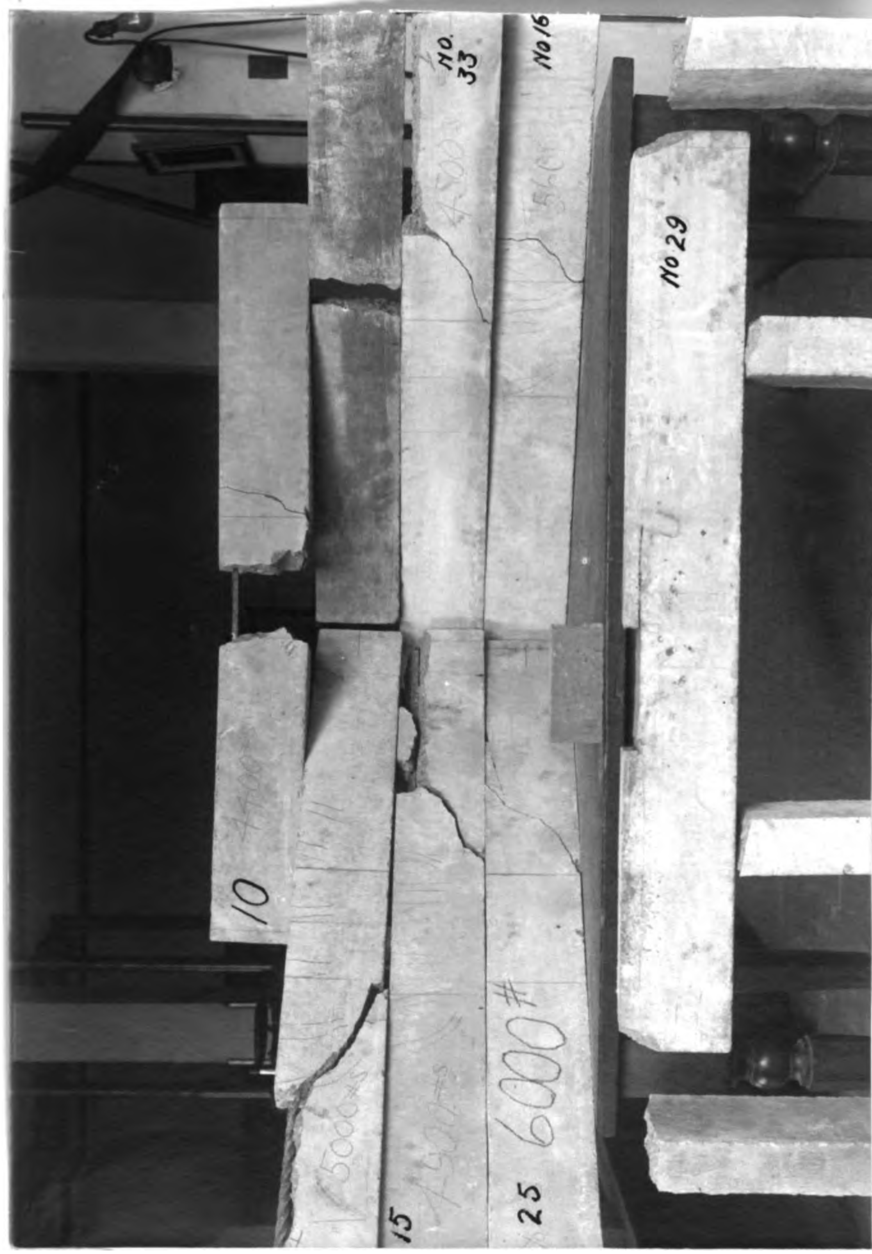


PLATE VII

PLATE VIII.

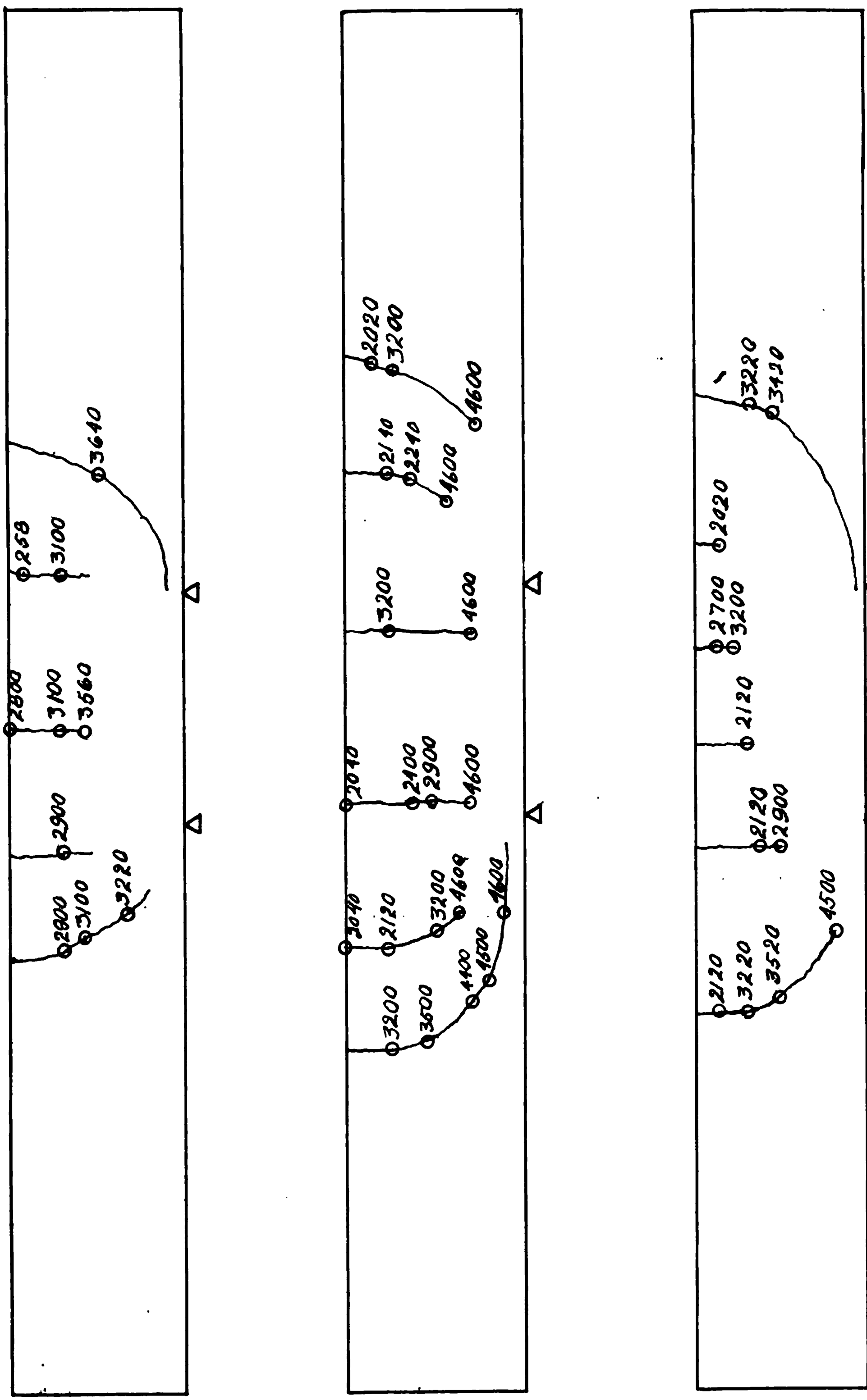


Fig. 8.- Showing Progression of Cracks on Side of Beam.

outside the section of uniform bending moment. Shortly before failure a longitudinal crack appeared on the top face, starting from one of the transverse cracks and extending gradually toward the end of the beam. All transverse cracks bent toward the knife edge as they extended down the side of the beam. The manner of final failure is plainly shown in Plates V, VII and VIII. Black lines on No. 6, in Plate V, show the position of the cracks and approximately down the side to which they had extended at failure.

Manner of Failure.

Just before failure, crack d (Figure 9) opened suddenly and crack f, starting from d began to open up. Occasionally crack d closed on top of the beam and a transverse crack g, a few inches nearer the end, would appear, connecting by an oblique crack with f. At failure, the concrete above the crack f would split off, generally breaking along crack c also.

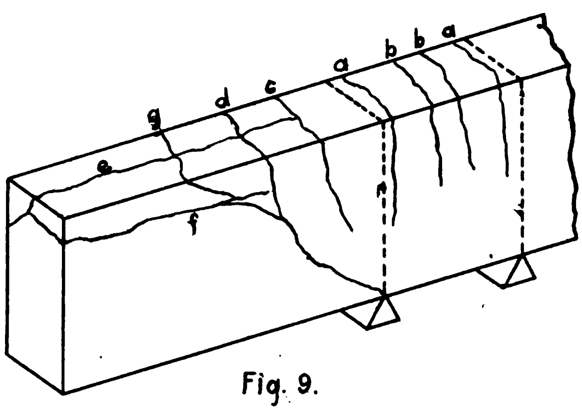


Fig. 9.

This mode of failure is shown very well on No. 61, Plate VIII. Invariably the crack at which failure occurred extended to the knife edge. As may be seen in the photographs, the position of the crack at which the failure started varies considerably.

Compression Failures.

Two beams, Nos. 10 and 11, failed by buckling on the compression side. No. 10 is shown on Plate VII. There were several cracks on the beam, as usual, but failure took place along the lines indicated in Figure 10. After failure the block above the rod was found to be entirely detached from the rod. The wedge below the rod on the tension side was pushed tightly against the rod and the concrete on the compression side was crushed and came to pieces in flakes, as indicated. The outlines of the compression break may be distinguished in the photograph Plate VII. The concrete on the tension side was broken out to determine the condition of the adhesion between the concrete and the rod. The adhesion was found to be nearly perfect.

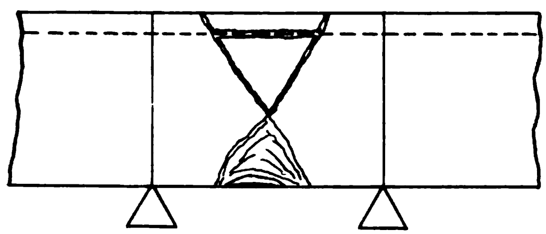


Fig. 10.

The usual mode of failure may be said to have been due to combined shearing stresses. No provision was made in the design of the beams for these stresses and no data was obtained regarding them further than observation of the manner in which failure progressed, as described above. On account of this failure in shear, the full strength of the

beams was not developed, with respect to the stresses caused by the maximum bending moment, and the breaking loads are much less than the beam could have carried had reinforcement been provided to care for the shearing stresses.

In a few cases, where beams were reinforced with smooth rods, the rods pulled loose from the concrete and failure took place along a previously developed crack in the same manner as unreinforced beams fail. An example of this failure is beam No. 16, shown on Plate VII.

Elongation of Reinforced Beams.

A discussion of the elongations obtained on the unreinforced beams of Group XIII has been given already. Elongations of the beams of Group XIII B, were obtained by double measurement. As stated these elongations checked closely with those computed from the deflections and we consider them accurate. On the grounds thatthe concrete under tensional stress in reinforced beams cracks at yield point, the yield point of the unreinforced beams occurs normally at failure, direct comparison can be made between the elongation of unreinforced beams at failure and that of reinforced beams at yield point.

Curves Nos. 57 and 72 obtained by "double measurement" show an elongation at yield point of 0.00022 and 0.00018 inches per inch respectively. Curves of beams Nos. 25, 28,

62, 45, 49 and 52 (Figures 13, 14 and 15), obtained by single measurement, show elongations of 0.00010, 0.00030, 0.00048 0.00021 and 0.00024 inches per inch, respectively. Curves Nos. 8, 15, 40, 38, 52 and 49, obtained with the extensometer, show elongations at yield point of 0.00022, 0.00019, 0.00018, 0.00025 and 0.00012, respectively. While wide variation was found by single measurement and extensometer, the general average by each method agrees quite closely with the elongations obtained in tests of beams Nos. 41, 42, 44, 57 and 72. Beam No. 72 is quoted from Table 4, of Series 2. Beams Nos. 57 and 62 are taken from the group which was stored in dry sand.

The tests quoted above include beams three months old and six months old. Nos. 41, 42 and 44 are unreinforced. No. 72 is reinforced with three round steel rods 3/32" in diameter. The others are each reinforced with a single steel rod. As shown, the elongation of the concrete at yield point was practically the same in all cases, and equal to about 0.00020 inch per inch, or 1 in 5000.

Considére's Tests.

In a series of articles on reinforced concrete,* M. Considere, Engineer of Bridges and Roads, gives a description of tests made by him on concrete-steel beams and a dis-

*Influence des Armatures Metalliques sur les Proprietés des Mortiéres et Bétons, Le Génie Civil, February 4-25, 1899.

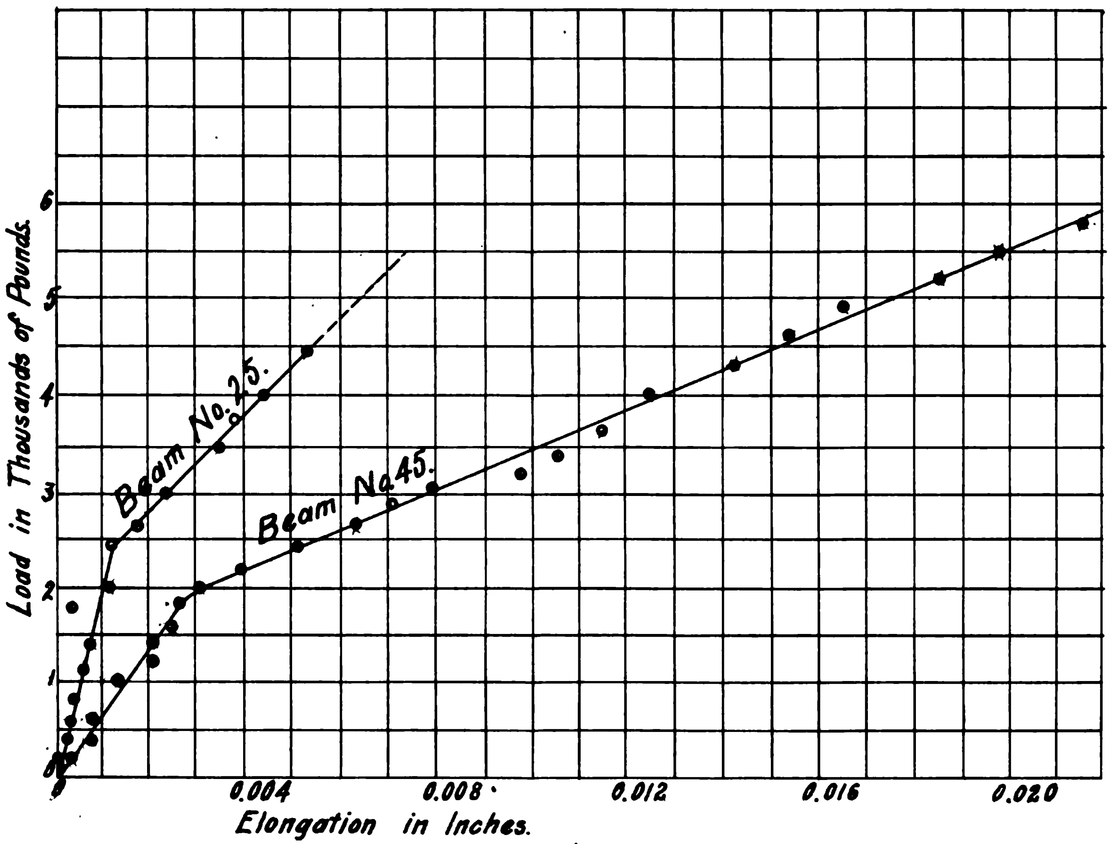


Fig. 2. Elongation Curves. Plotted as Measured over the 8" Section.

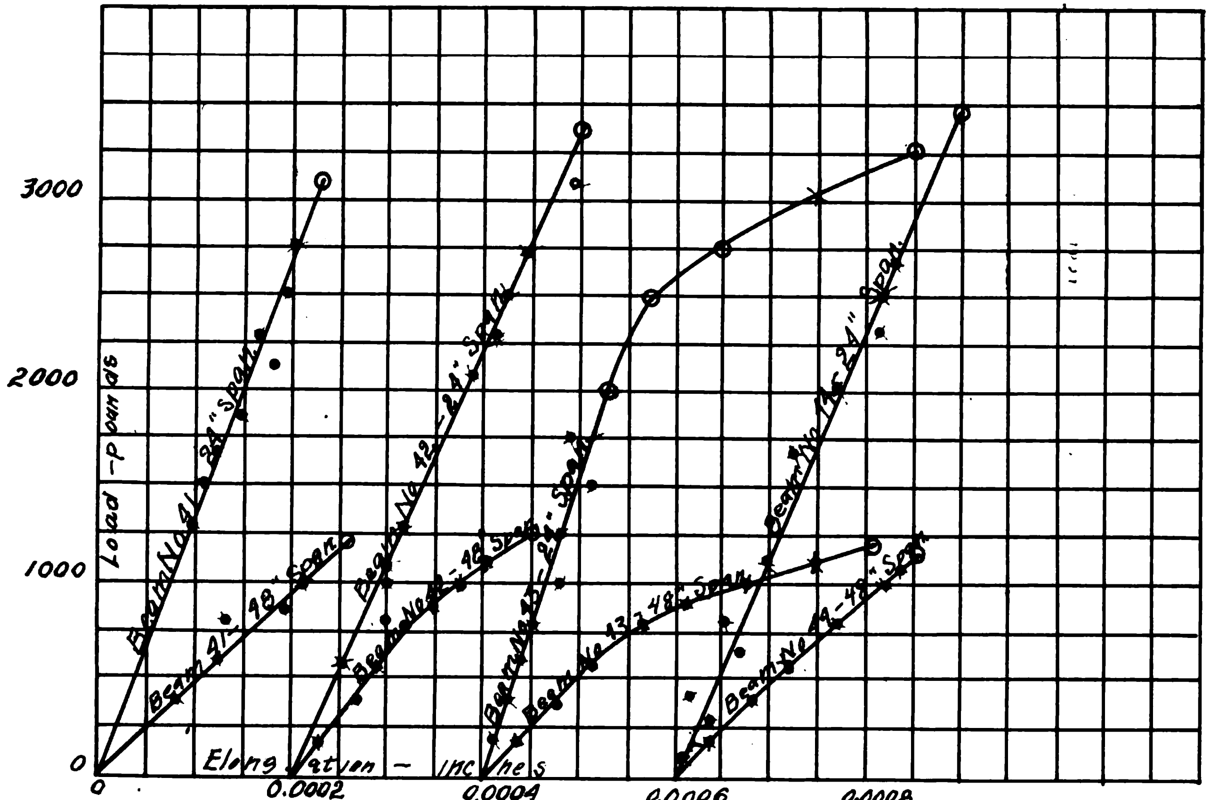


Fig. 12.- Elongation Curves for beams of Group XIII

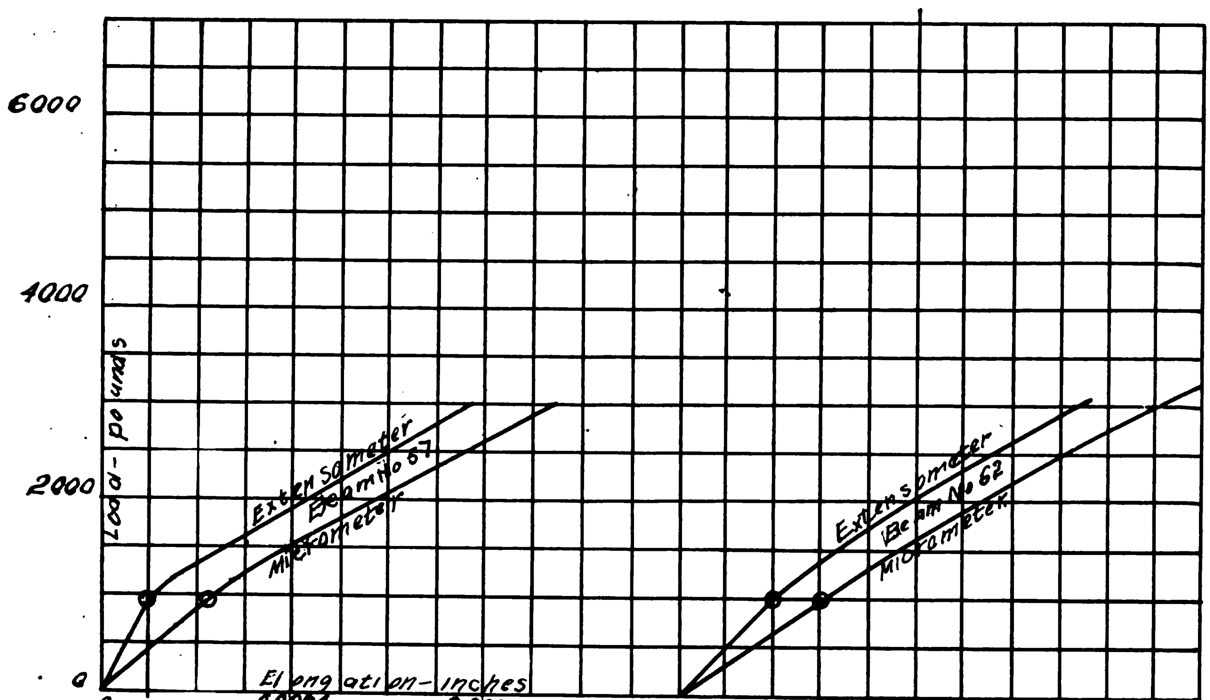


Fig. 13.- Gives a Comparison of Elongation Curves Obtained with Micrometer and Extensometer.

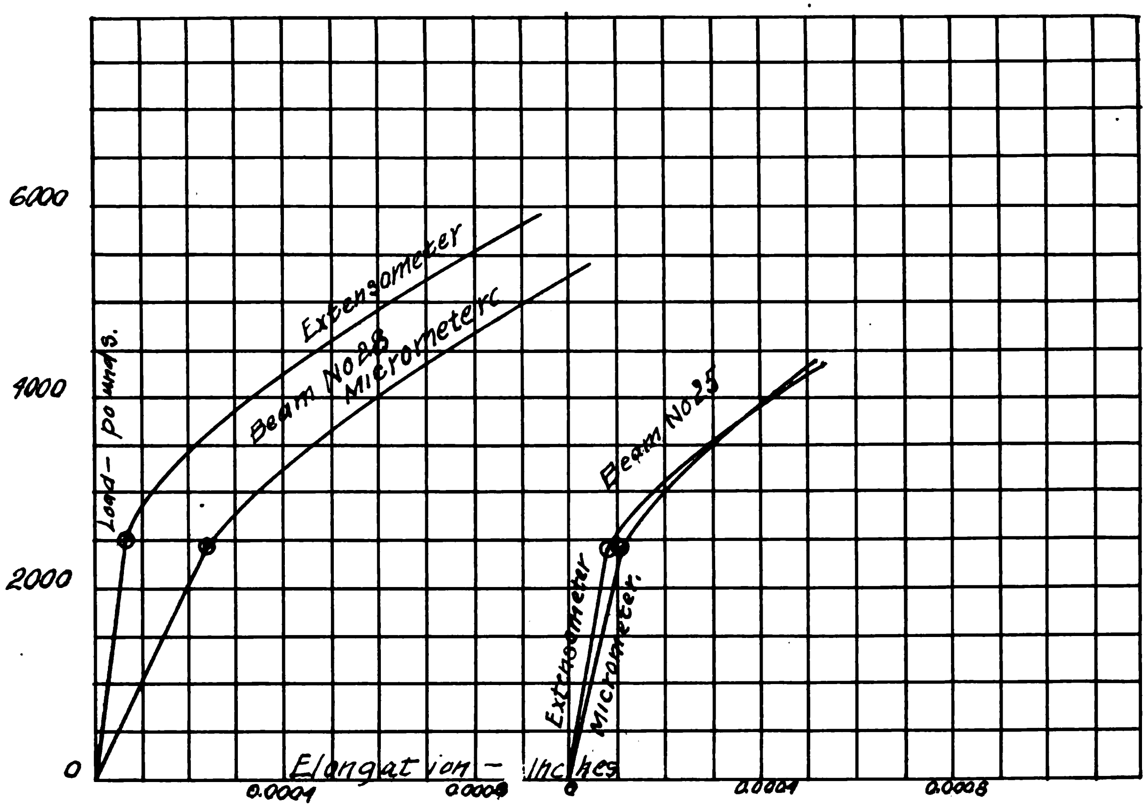


Fig. 14.– Gives a Comparison of Elongation Curves Obtained with Micrometer and Extensometer.

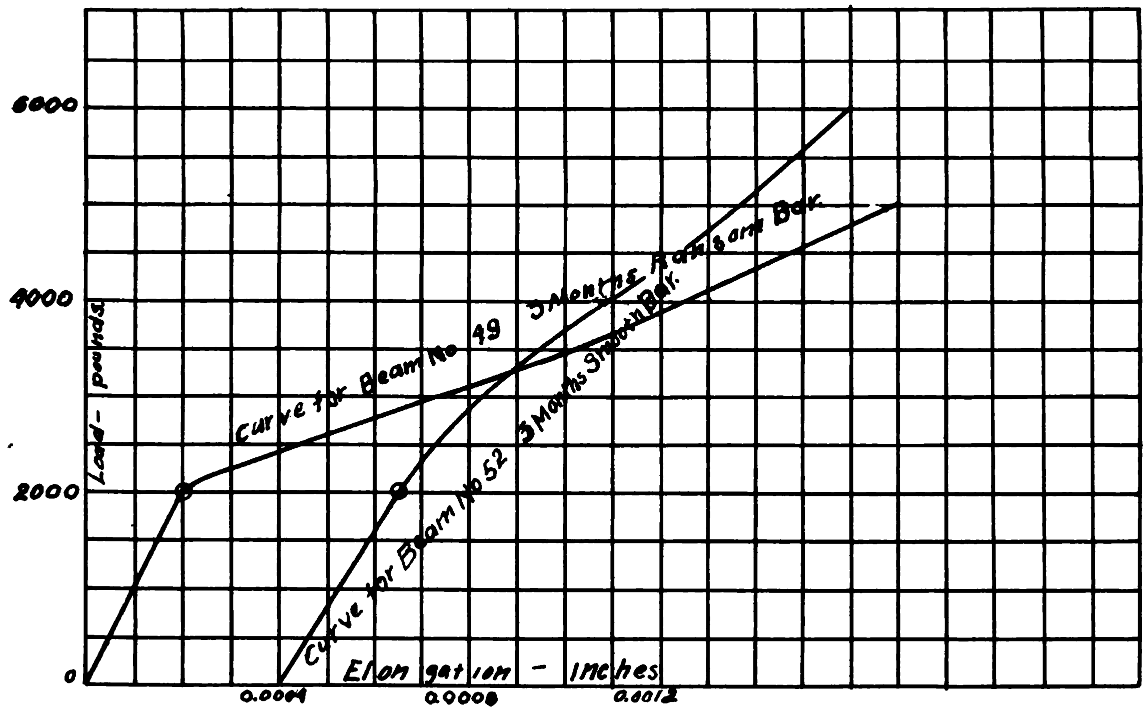


Fig. 15.- Elongation Curves for 2 - 4"×6"×48" Beams

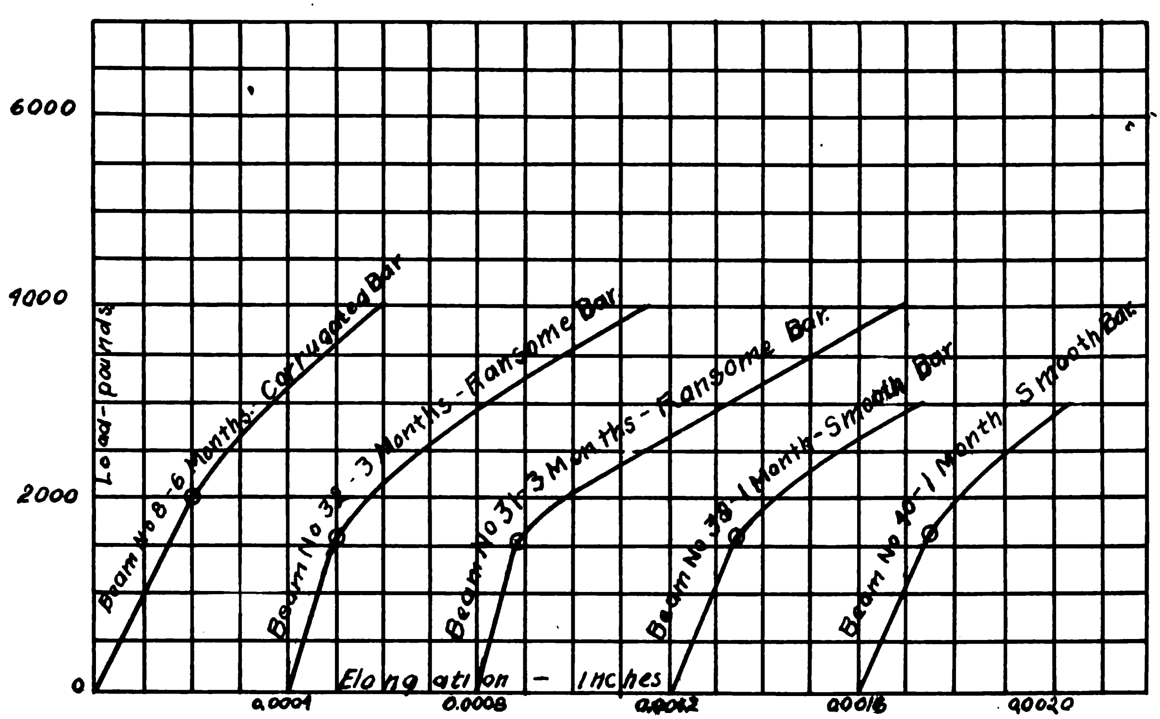


Fig. 16.- Elongation Curves for 3"×6"×48" Beams.

cussion of the results of the tests.

The tests were made as follows:- The beam was set up in a socket in the testing table. A sleeve, to which was attached an arm, was set over the upper end. The arm carried a pan in which weights were placed. The length of the lever arm could be varied by moving the pan along it. The length of beam in uniform bending moment could be varied by changing the position of the sleeve. This arrangement is shown in Figure 17.

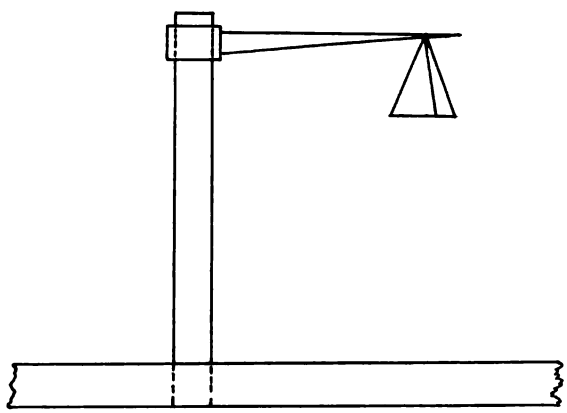


Fig. 17.

The elongation and compression of the concrete in the beam, subjected to flexural stresses, was measured, apparently, by a mirror apparatus, throwing a spot of light on a scale. No description of this apparatus is given.

With regard to the bending moments applied and the resulting elongations obtained in the tests of the typical beams Nos. 31 and 34, M. Considére says:

Results of Tests.

"The unreinforced beam No. 31 had a section 60 m.m. high by 61 m.m. wide; it broke after having supported for

some minutes a bending moment of 11.48 kgm. (835 ft. lbs.) which produced, at its application, a shortening of extreme compressed fibers of 0.131 m.m. and an elongation of extreme fibers in tension of 0.201 m.m. per meter.

"The luminous spot, the displacement of which indicated the amount of elongation, could be followed only to rupture, which occurred after an elongation of 0.266 m.m. (about 1 in 4000).

"The test of reinforced prism No. 34 has been carried farthest and offers most interest. It has the section opposite, the three wires having a diameter of 4.25 m.m. and their centers being at 7 m.m. from the surface of the prism.

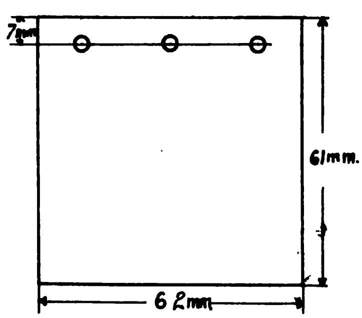


Fig. 18.

"We will give in detail the figures relative to this beam only, but the results of the tests of the other beams, reinforced with wires of 4.25 m.m. and 1.9 m.m. were almost absolutely identical. Those of the beam reinforced with 7.7 m.m. iron differed only because of the lower elasticity of the rolled iron and the greater distance from the center of the bar of the bar of iron to the surface of the prism, the result of its greater diameter.

"The bending moment on beam No. 34 was carried up to

78.68 kgm. (570 ft. lbs.) without causing rupture. Then to study the effect of repeated deformations, we submitted the beam to 139,042 repetitions of bending moment, varying from 34.58 kgm. to 55.58 kgm., separated to the extent of returning to the position of equilibrium.

"After this double test, the beam seemed intact in all that part between the fastenings, and yet the mortar of the face submitted to tension had undergone, in the first bending, an elongation of 1.98 m.m.; that is to say, almost twenty times more than the elongation of 0.10 m.m. which the corresponding mortar could not support without breaking, and had then supported, 139,042 times, elongations varying from 0.545 to 1.270 m.m. per meter.

"To determine with certainty if the extreme fibers were not cracked, we have detached them, by means of a sand saw, from the reinforcement and also from the body of the beam, and have determined that, while there were, at two points, superficial cracks 2 to 4 m.m. long, the mortar was elsewhere perfectly intact. In spite of the fatigue resulting from the sawing, we were able to detach from the beam and reinforcement, rods 15 m.m. x 20 m.m. having a length of 80 to 200 m.m.; that is, more than half the length comprised between the fastenings. These rods were tested by bending and gave a resistance up to 22 kg. per square centimeter.

"It is, therefore, well demonstrated that, in nearly

the whole mass, the mortar of the extreme fibers of beam 34, which had undergone an elongation twenty times more than that which we supposed must produce failure, not only was not disintegrated, but remained capable of showing a considerable resistance, and somewhere about that of new mortar."

Comparison of Considére's Beam No. 34 with

Beam No. 29.

It is worthy of note that he found "superficial cracks at two points" on that portion of the concrete which had undergone tensional stress. In the next paragraph he says, "In nearly the whole mass the mortar was intact." This would suffice to throw into doubt any statement that the mortar elongated twenty times as much as unreinforced concrete in tension. Referring again to our beam No. 29, at a time when one crack only was visible, and that only by means of microscope, a method similar to Considére's, that of sawing out the extreme fibers in tension, proved that there were two cracks practically across the beam. And the tests of those pieces which were sawed out showed the presence of two incipient cracks. Moreover, these were all three inches, at least, apart, and judging from the results of the other tests no more cracks would have appeared among them. If a crack which could be seen only with a high power microscope, and then for a length of only one-fourth inch, extends

across the beam, what else would be expected of a crack, which could be seen easily and which was the same length?

In view of these statements, it would seem that M. Considére ought to have said that, with the bending moment which was imposed upon his beam, the face of the beam which was under tensional stress elongated twenty times as much as unreinforced concrete in direct tension.

M. Considére's results will be taken up later in connection with the results of our tests, Series 2.

Deflection of Reinforced Beams.

Method of Measurement.

Figures 19 and 20 show typical load deflection curves taken from each group of beams. Deflections of beams in Groups I to VI, inclusive, were measured by direct reading of the movement of a fine silk thread over a polished scale at the middle of the beam. By this means deflections could be obtained to the nearest 0.005 inch. The hook micrometer device was used to measure the deflections of the beams in the remaining groups of this series. This arrangement assured an accurate measurement to 0.0005 inch. A comparison of the curves from the first six groups, with those of the other groups, shows a close agreement of deflections by the two methods. This is especially true of the yield point.

Comparison of Beams with Varying Reinforcement.

In Figure 22 are given average curves of load and deflection of beams in Groups III, IV and V, all three-months old. Beams of Group III are reinforced with smooth bars, of Group IV with Ransome twisted bar and of Group V with Johnson corrugated bar.

Deflection.

It will be noticed that the yield points on these three sets of curves are almost identical, the difference in

extreme loads being only 100 pounds and between extreme deflections only .005 inch. Above the yield point the curves diverge a little, the beams of Group III being somewhat stiffer. However, Groups IV and V run very close together and almost exactly parallel after a load of 2500 pounds is reached.

Elongation.

Elongations of the beams of these three groups were measured by means of extensometer alone. On account, possibly, of friction in the connections and fastenings, also because of inertia effect of the arms, the small elongations produced before reaching the yield point, which were actually never more than .004 inch, were not recorded in most cases correctly and, therefore, the curves, as regards elongation, are of no value from a quantitative standpoint, especially at the yield point. In all cases, however, the elongation curves showed a distinct bend at a point corresponding closely to the yield point, as shown by the deflection curve. Thus it proves its distinct value as detecting this yield point. Mr. Hartman says, "In a tension test this machine has shown its worth, when care was used in handling the same. Though correct in principle the machine, as built, has structural faults which can be remedied easily."

At failure the loads carried by beams of Group III were much larger than those carried by beams of Groups IV

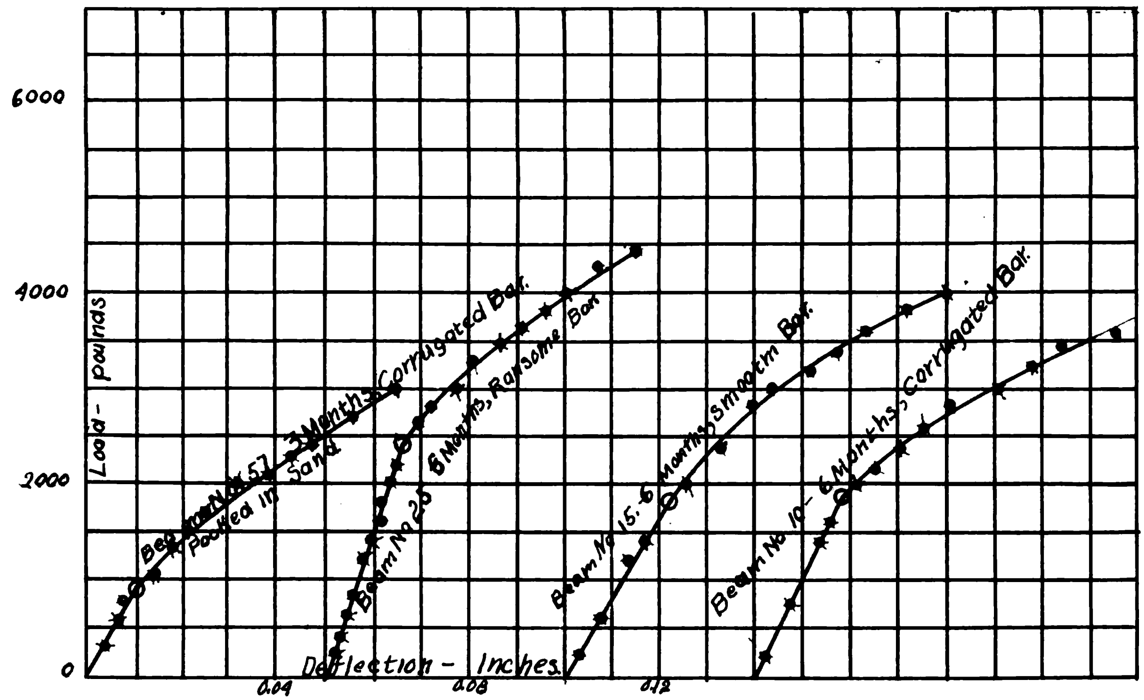


Fig. 14.- Showing Typical Deflection Curves.

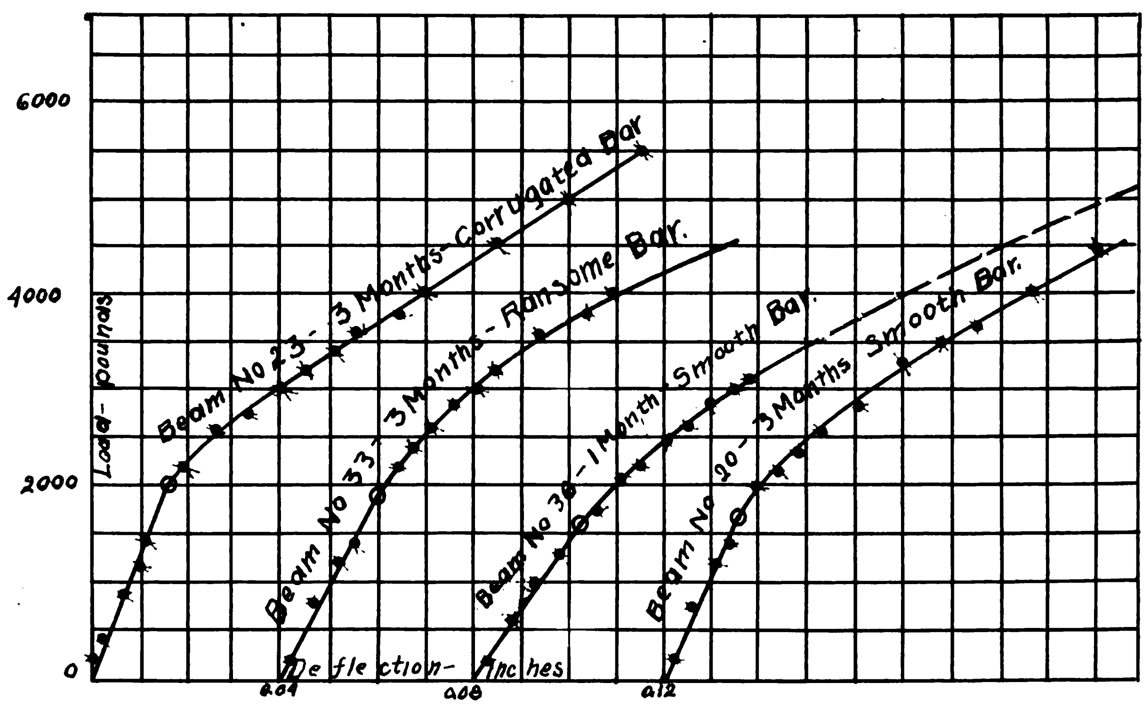


Fig. 20.- Showing Typical Deflection Curves.

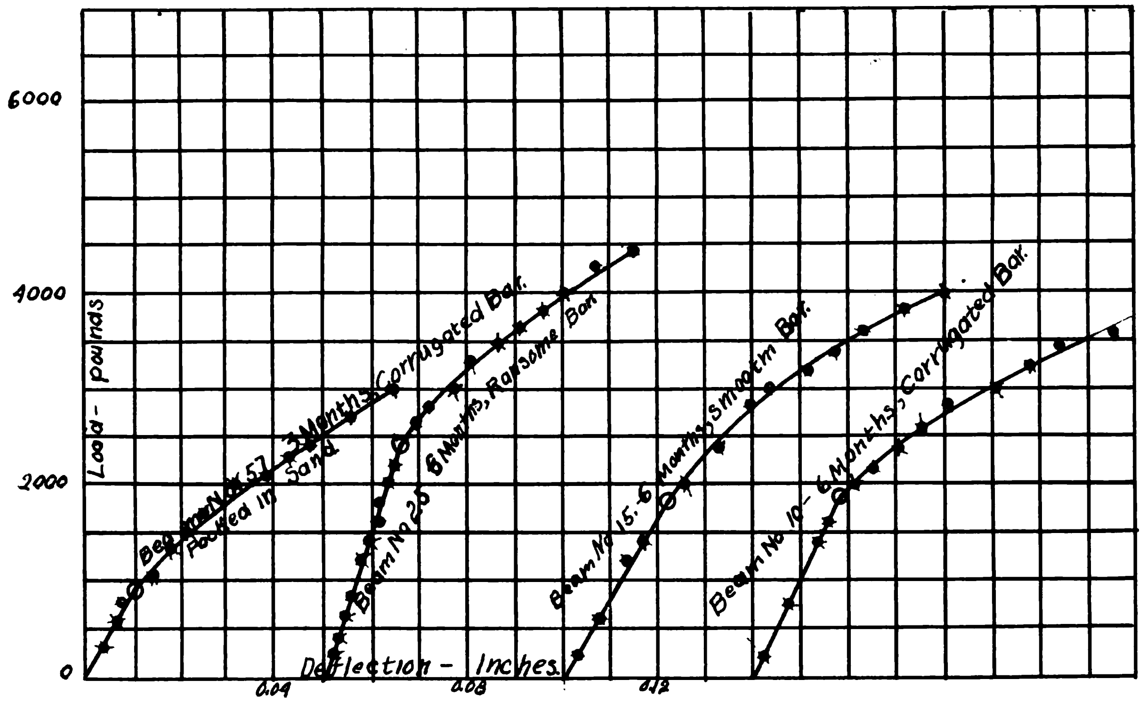


Fig. 19.- Showing Typical Deflection Curves.

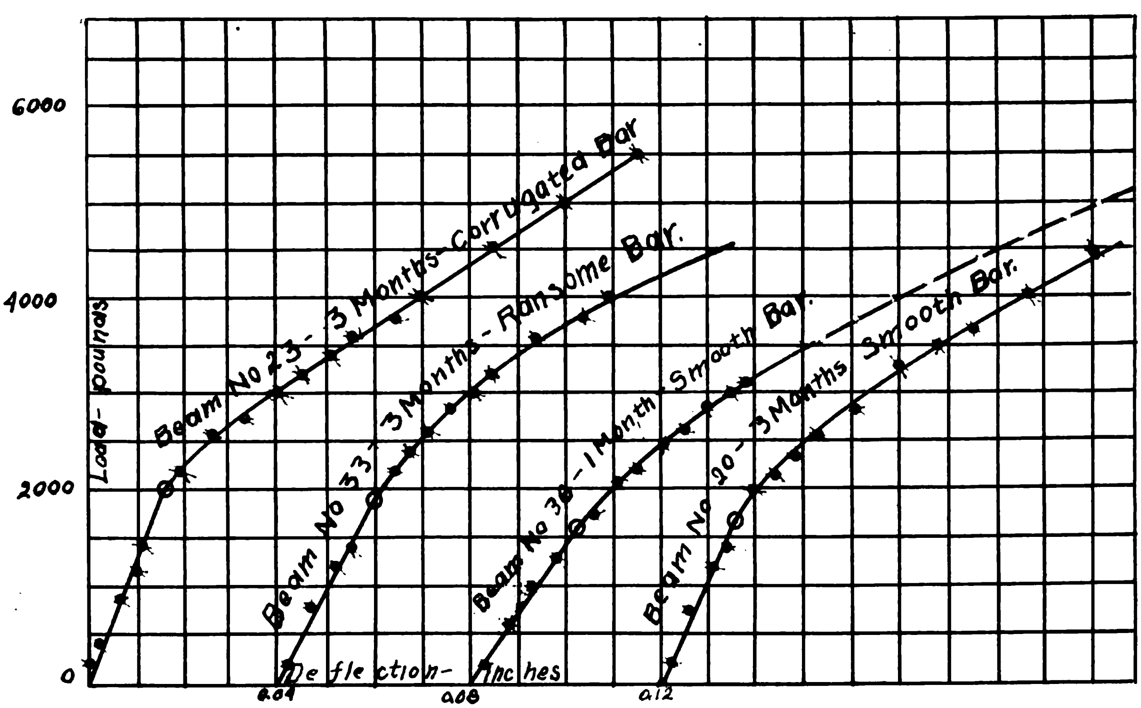


Fig. 20.- Showing Typical Deflection Curves.

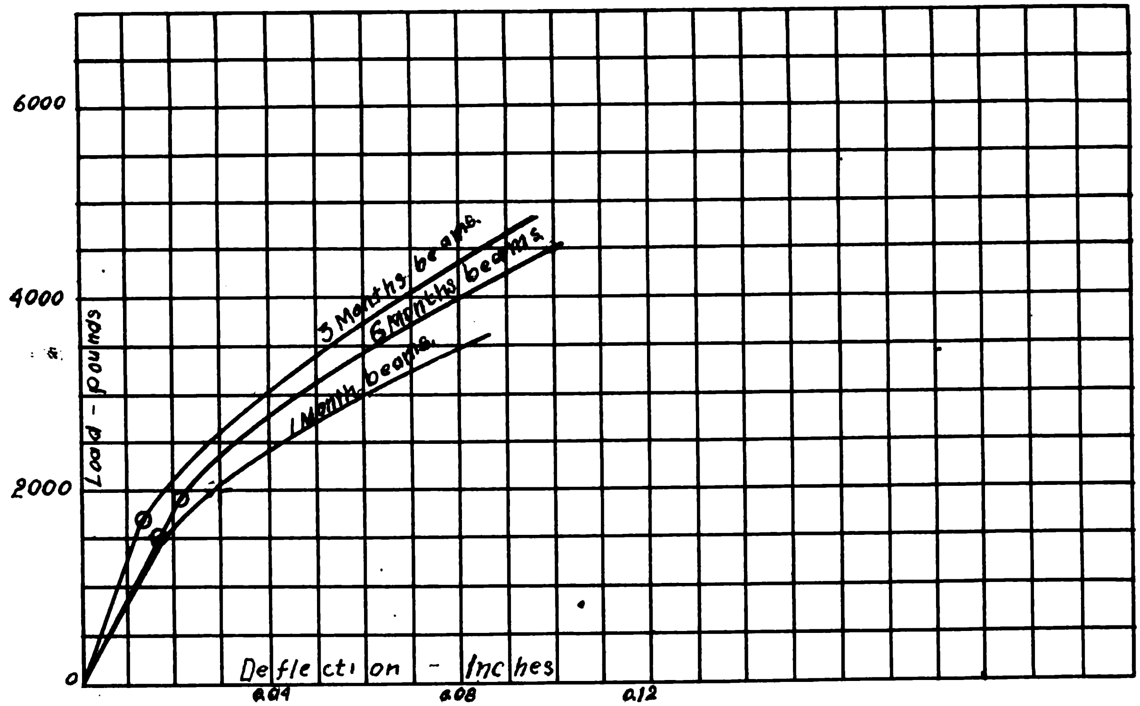


Fig. 21.—Average deflection curves of Beams of Groups II, III and VI Showing the Effect of Age on Steel Concrete Beams.

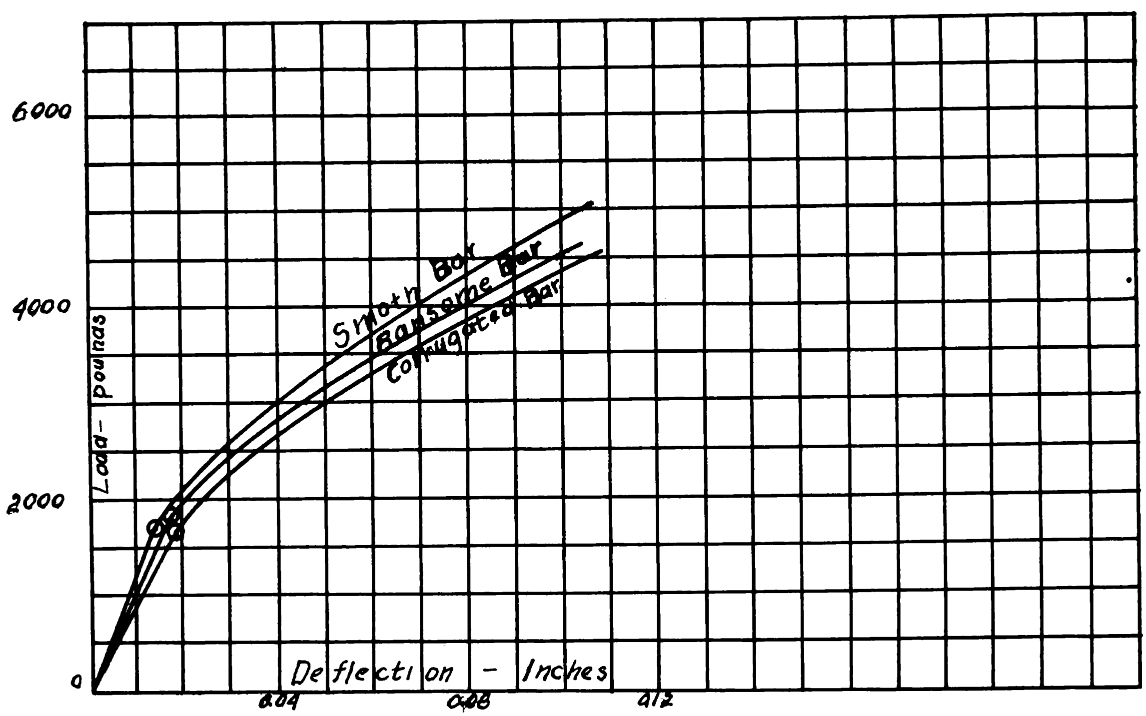


Fig. 22.—Average Deflection Curves of Beams of Groups II III and IV Showing a Comparison of Beams with different Bars.

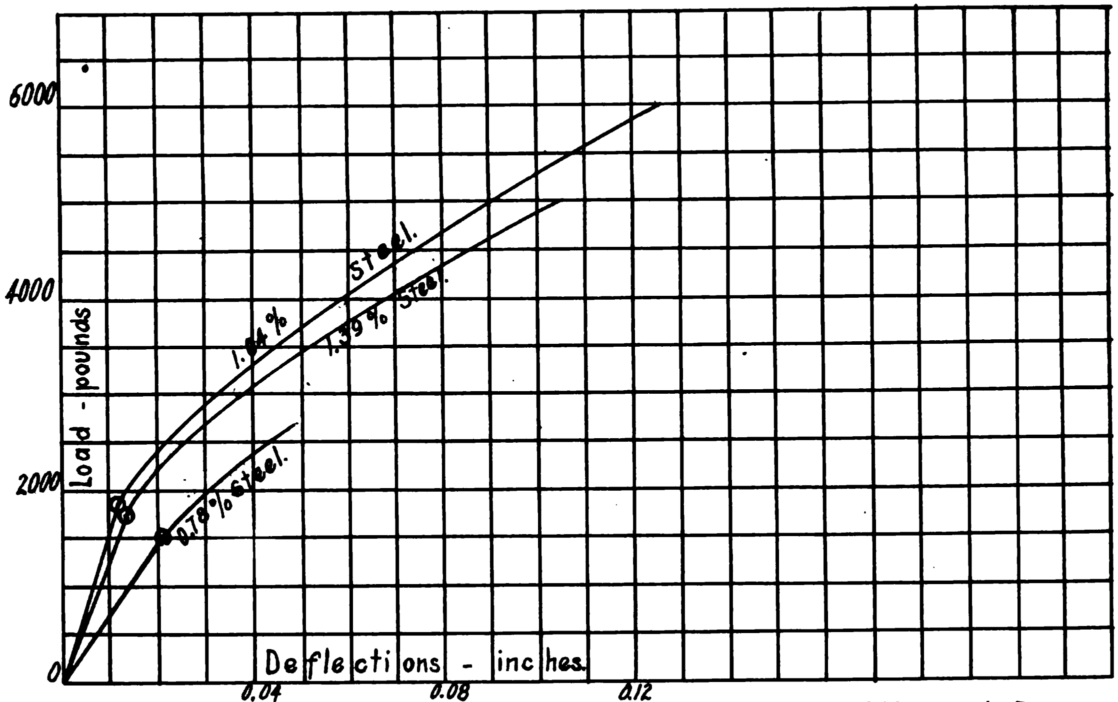


Fig. 23.- Deflections of Beams Reinforced with Different Percentages of Steel. All Beams of same Height and Span.

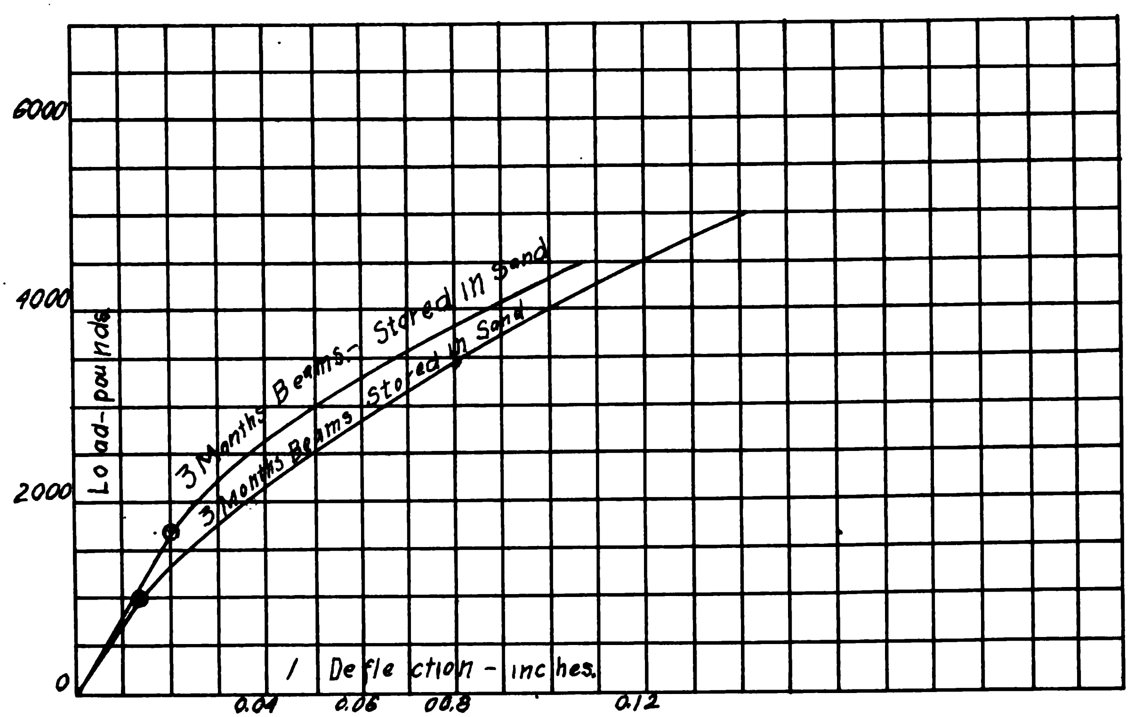


Fig. 24.- Showing Average Deflection Curves of Beams Stored in Sand and Water.

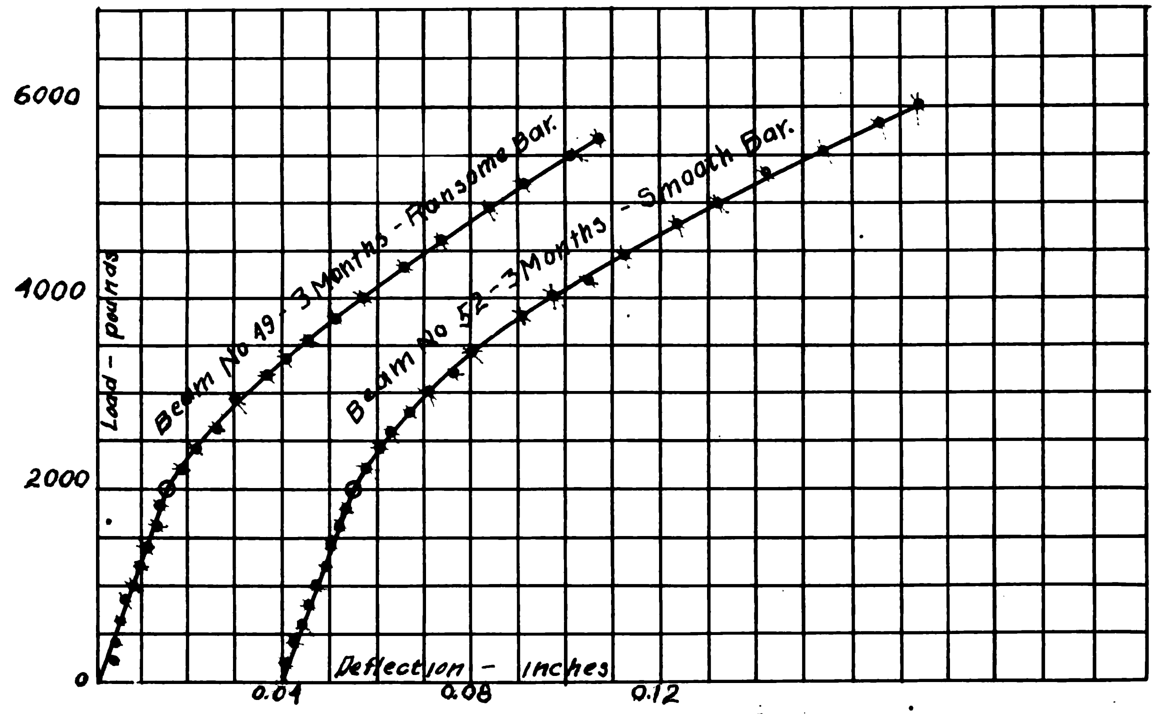


Fig. 25.- Showing Deflection Curves of 4"×6"×48" Beams.

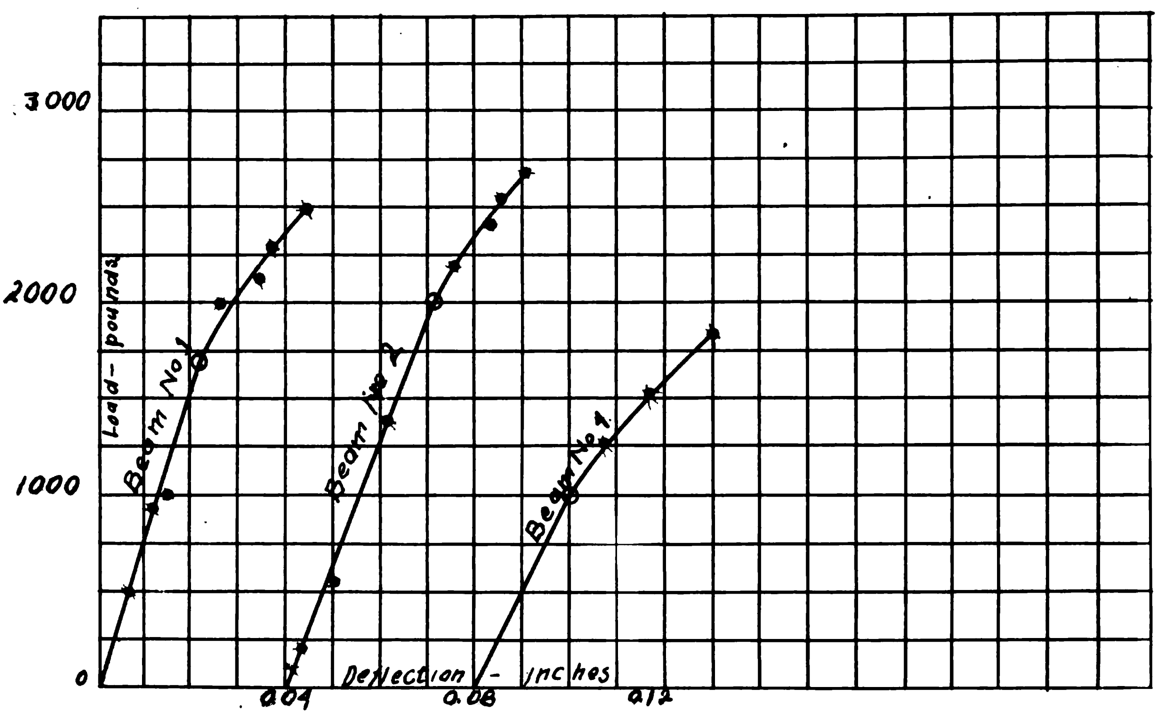


Fig 26.- Showing Deflection Curves of Beams Reinforced with 3/8" Smooth Bars.

and V. In Group VI, of beams six-months old, reinforced with smooth bars, the loads at failure were much lower, due to the fact that the rods were pulled loose from the concrete. In Group VII, of six-months beams with corrugated bars, the load at failure varied from 5000 to 6000 pounds and in Group VIII, of six-months beams with Ransome bars, it varied from 5800 to 6000 pounds. Originally there were five beams in Group VIII. When tested three of these beams were found to be defective. There was no adhesion between the concrete and the steel. The concrete appeared as though too little water had been used in gauging the mortar of which it was made. In the other groups mentioned, four to six beams have been considered.

Comparison of Beams of Various Ages.

In Figure 21 are shown deflection curves of beams, from Groups II, III and VI, reinforced with smooth bars, and one-month, three-months and six-months old, respectively. In comparing these groups the yield points show a rather wide and quite peculiar variation. The yield point of the six-months beams occurs at about 1900 pounds with a deflection of 0.022 inch. The curve of the one-month beams coincides with that of the six-months beams almost exactly, up to the yield point, which occurs at 1500 pounds load with a deflection of 0.019 inch. The one-month beams then deflect much more rapidly, and at a load of 3500 pounds show a de-

flection of 0.08 inch, while the six-months at the same load show a deflection of 0.06 inch. The three-months beam, on the contrary, are much stiffer than either of the others, the yield point occurring at a load of 1700 pounds with a deflection of only 0.014 inch. Above the yield point, however, its deflection is practically the same as that of the six-months beams, the curves being very nearly parallel. The loads at yield point, of these three groups, show a slight increase with the age, as might be expected. The load at failure shows the same variations as the deflection, breaking load for Group II being 4500 to 5200 pounds, for Group VI from 4200 to 5600 pounds, for Group III from 4200 to 6500 pounds. In Group II, three of the six beams failed on account of slippage of the rod; in Group III, three out of five and in Group VI, one out of five. The other beams in these groups failed in shear, and these only, having been considered above. It may be said, however, that in Group III the rods slipped in two of the beams at loads of 5600 pounds and 6000 pounds.

The Effect of Varying Percentages of Metal.

Figure 23 shows two typical deflection curves, one from Group III and one from Group XII. The percentage of metal, with respect to cross-section of concrete, in beams of Group III, was 1.38%, in those of Group XII, 1.04%. The

difference at yield point is very slight, the beams of Group XII being slightly stiffer and stronger. At failure, also, beams of Group XII were slightly stronger. As all these beams failed in shear, this excess of strength is due directly to greater amount of concrete in cross-section of the larger beams.

Comparison of Wet and Dry Beams.

In Figure 24 are given two curves from Groups V and IX, respectively. These two curves show a marked difference. At the yield point of beams in Group IX, the load is only about 60% of the load at yield point of beams in Group V. The deflections, however, are nearly the same, being .013 for Group V and .014 for Group IX. Group IX is somewhat stronger at failure than Group V, the breaking loads being 4600 to 6400 pounds and 4400 to 5500 pounds, respectively.

Comparison of Different Styles of Bars.

As regards the comparative merits of the bars used for reinforcement, no data has been obtained in these tests, as between twisted and corrugated bars, which would warrant any statement. Mention has been made of the fact already that the smooth bars, in several cases, slipped before the full strength of the beam had been developed. Nothing of this kind occurred with either the twisted or the corrugated bars.

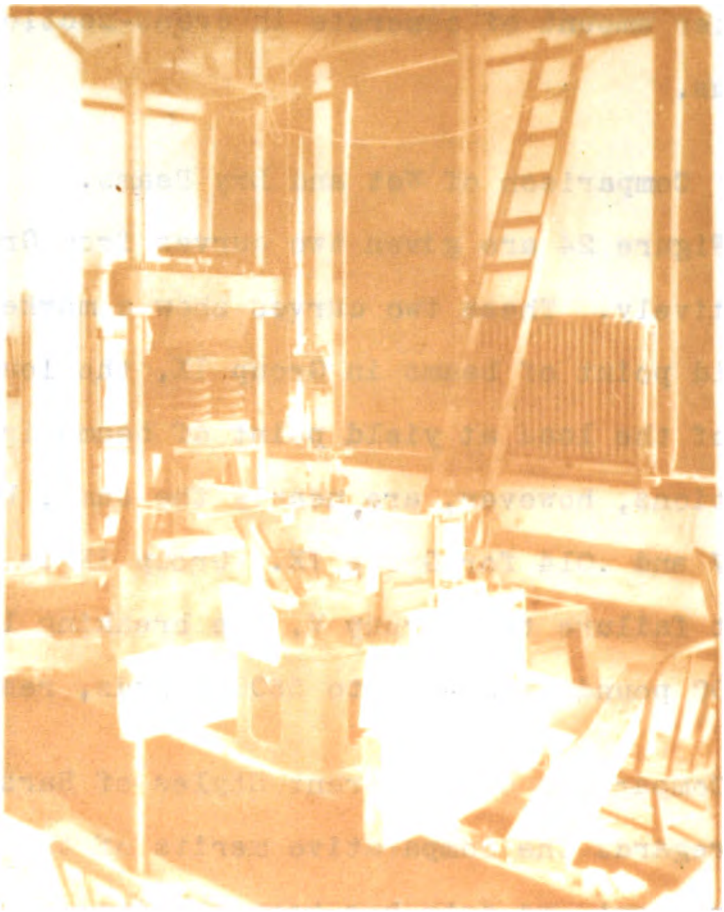


PLATE IX.

Time Tests of Beams Nos. 57 and 58.

Time tests have been made on beams numbers 57 and 58 of Group IX. There was one change made in the apparatus for these tests. This was the insertion of a set of springs between the knife edge and the cross-head of the testing machine, as shown in Plate IX. These springs were put in to take up any deflection which might occur in the beam and thus keep the load constant. A little difficulty was experienced in keeping the load constant which was due to a slight leakage of oil from the cylinder. This leakage caused a gradual decrease in the load, but with constant attention the load was kept from any excessive variation.

As may be seen from the curves, Figure 19, the beam was loaded gradually to 3000 pounds. The yield point occurred at a load of 900 pounds with an elongation of 0.000231 inches per inch and a deflection of 0.01 inch. At 3000 pounds the elongation was 0.00155 inches per inch and the deflection was 0.064 inch. The beam was then gradually unloaded. At zero load the elongation was 0.00045 inches per inch and the deflection was 0.026 inch. The springs were then inserted and the load raised to 3000 pounds again. This time the micrometer elongation was 0.0012 inches per inch, the extensometer elongation was 0.0010 inches per inch and the deflection was 0.06 inch. From this time on the load was maintained between 3000 and 3100 pounds for twenty-

seven days. As may be seen from the curve, Figures 27, 28 and 29, both elongation and deflection increased gradually for about twenty days then reamined practically constant. At the end of the twenty-seven days the beam was loaded to failure, which took place at a load of 6420 pounds.

Beam number 58 was loaded to 2000 pounds and left for a second time test. The yield point occurred at 1100 pounds. This increase in strength was due in part to the fact that the beam was a month older when tested. The day after the time test was begun the load was suddenly raised to 2560 pounds, causing an increase in elongation of 0.00016 inches per inch. After this the load decreased steadily to 2280 pounds in ten days but the deflection and elongation remained practically the same. On the tenth day the machinery was tampered with and load raised to 2400 pounds, with an increase of elongation amounting to 0.0005 inches per inch. From that time the load varied between 2400 and 2300 pounds. The deflection and elongation remained practically constant. This beam has not been tested to failure.

Formulas used in Computations.

As a means of comparison of some of the various formulas used in the design and review of steel concrete, computations have been made to determine the position of the neutral axis and load at yield point and failure. Formulae

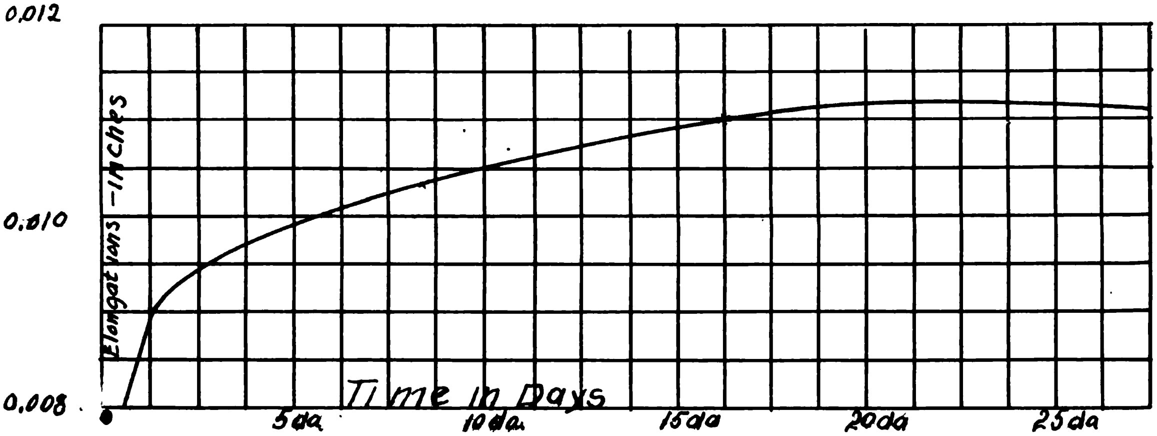


Fig. 27.- Time Elongation Curve, with Extensometer, of Beam No 57.

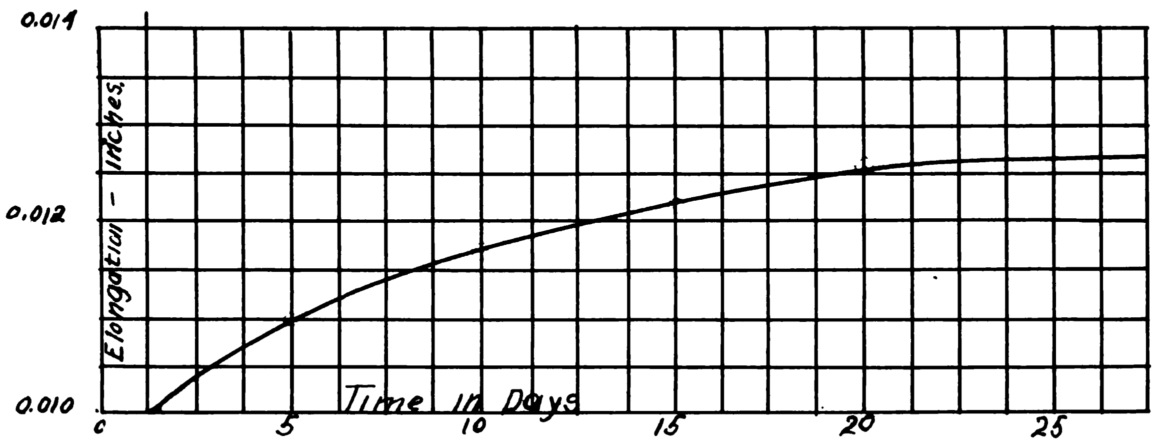


Fig. 28.-Time Elongation Curve, with Micrometer, of Beam No. 57.

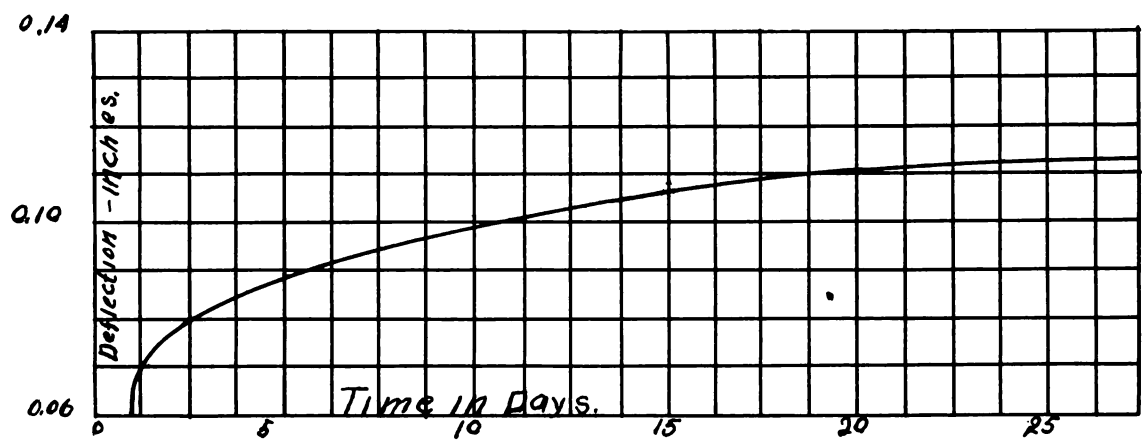


Fig. 29.-Time Deflection Curve. of Beam No. 57.

developed by Edwin Thatcher, C. E., A. L. Johnson, C.E. and Professor Hatt, of Purdue University, have been compared. A method, given by the late Dean J. B. Johnson in his book on The Materials of Construction, which is applicable to the determination of the neutral axis and load at yield point, has been used.

Assumptions.

In all of these formulae no account is taken of initial stresses due to expansion or contraction of concrete while setting. It is assumed (a) that cross-section, plane before bending, remains plane after bending; (b) that the applied forces are perpendicular to the neutral plane of the beam; (c) that there is no slipping between concrete and reinforcement. Professor Hatt also assumes that the value of E, obtained in simple tension and compression tests, will apply to the material under stress in the beams. "With reference to the relation between simple tests and flexural tests, it may be expected that the material on the compression face of the beam will bear a greater fiber stress than that indicated in a direct compression test, for these outer fibers are supported by material which has not reached its maximum strength."*

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*Tests of Concrete-Steel Beams by Prof. W. K. Hatt, Engineering Record, Vol. 46, p. 605.

Table No. 3.

Table No. 3 shows loads at yield point and breaking point for the highest, lowest and average beams in each group. Also the loads and positions of the neutral axis at these points, as computed by the various formulae.

J. B. Johnson's Method.

By Dean Johnson's method the neutral axis at yield point is placed at what would be the center of gravity of the section if the area of the steel were transformed into its equivalent in concrete by building out wings on each side of the beam, the total area of which would be as many times less one as great as the steel area, as the modulus of the steel is times as great as the modulus of the concret The distance y_2 of the neutral axis, from the compression face of the beam, is given by the equation $y_2 = \frac{h}{2} + \frac{e}{1 + \frac{bh}{a} \cdot \frac{Ec}{Es}}$

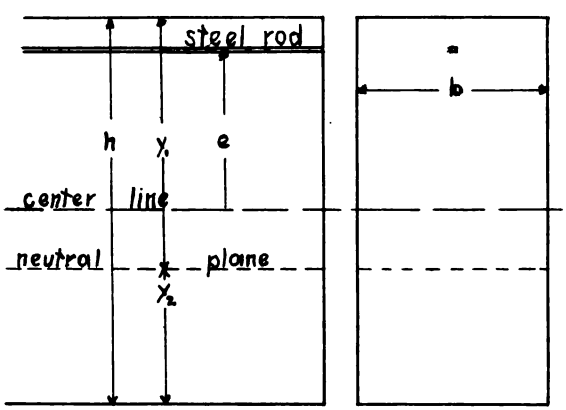

Fig. 30.

Where h = height of beam,

e = distance center of beam to center of bar,

b = breadth of beam,

Ec and Es = modulus of elasticity of concrete and steel respectively.

Having the neutral axis the moment of resistance is obtained by the formula

$$M_o = \frac{S\,I}{C}$$

where S is stress on extreme fiber, in tension,

I = moment of inertia,

C = distance neutral axis to extreme tension fiber.

The modulus of elasticity of the concrete used in these and following formulas is the average modulus obtained from the compression test described elsewhere in this paper. The modulus thus determined was 1,300,000 = Ec. The modulus of elasticity of the steel E was assumed to be 30,000,000. The extreme tensional fiber stress was found to 675 pounds per square inch. This was determined from the results of tests of the unreinforced beams of Group XIII, in the following manner. From the fundamental formulas of mechanics, the bending moment M of the center of a beam supported at the ends and loaded with equal concentrated loads P_1 and P_2 symmetrically, placed about the center, equals $\frac{Pa}{4}$, ?

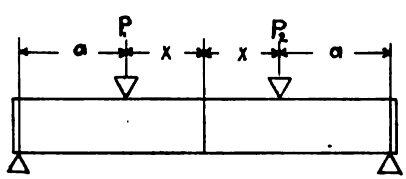

Fig. 31.

Where a = distance of P from the support. The moment of resistance $M_o = \frac{SI}{c}$. Since $M = M_o$, $\frac{Pa}{4} = \frac{SI}{c}$; therefore $S = \frac{Pac}{4I}$. This gives the ultimate tensional

fiber stress at the surface of the unreinforced beam at failure.

The yield point and the breaking point of the unreinforced beam coincide. In the case of the reinforced beam, the concrete fails in tension at the yield point. For this reason the value of S, determined above, may be used in computing the moment of resistance and load at yield point in case of reinforced beams. However, this assumption holds only when I and c are derived by the method of replacing the steel with its equivalent area of concrete.

Professor W. K. Hatt's Formulas.

Professor Hatt's formulas* were also used to compute the neutral axis, load at yield point and the load at failure. For the yield point, the position of the neutral axis is given by the equation

$$\frac{2}{3}\, x^2 n = \frac{2}{3}\,(1-x)^2 + pm\,(u-x)$$

The moment of resistance is given by the equation

$$M_0 = tbh^2\left\{\frac{5}{12}\cdot(1-x)^2 + \frac{5\,Nx^3}{12\,(1-x)} + p\left(\frac{u-x^2}{1-x}\right)\cdot m\right\}$$

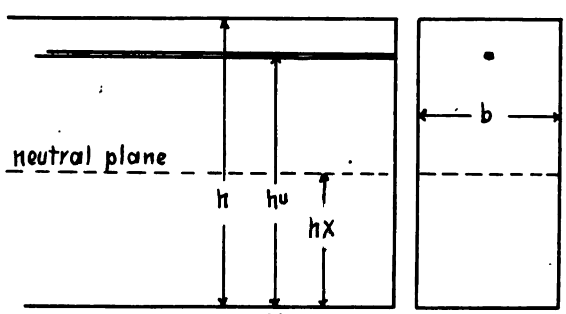

Fig 32.

In these equations

hu = distance compression face of beam to center of bar,

hx = distance compres-

*Engineering Record, Vol. 46, p. 605.

sion face of beam to neutral axis,

h = height of beam,

$n = \frac{E_c}{E_t}$ E_c and E_t equal modulus of elasticity of concrete in compression and tension respectively,

$m = \frac{E_s}{E_t}$ in which E_s = modulus of elasticity of steel,

M_o = moment of resistance,

t and c = ultimate strength of concrete in tension per square inch, and in compression per square inch.

b = breadth of beam.

Since the concrete is cracked above the yield point, the tension effect of the concrete must be neglected in the computations for neutral axis and load at failure, and the equations then become,

For neutral axis, $\frac{2}{3}cx = pf$,

For resisting moment, $M_o = bh^2\left\{\frac{5}{12}cx^2 + pf\,(u-x)\right.$,

f = stress in the steel.

The constants t, E_t and E_c are determined by direct tension tests. From the tension and compression tests, described in the first part of this paper, the constants t and E_c were assumed to be 350 pounds and 1,300,000 pounds, respectively. In the absence of any data from which to determine E_t, it was assumed to be equal to E_c.

The results obtained by the two methods just described correspond closely and agree with the average results obtained from tests.

The load at failure has also been calculated by formulas given by Messrs. Edwin Thatcher and A. L. Johnson. Johnson's formulas* give for the neutral axis the equation

$$y_2 = \frac{2\ F\ E_c}{3\ f_c\ E_s} \cdot y_1$$

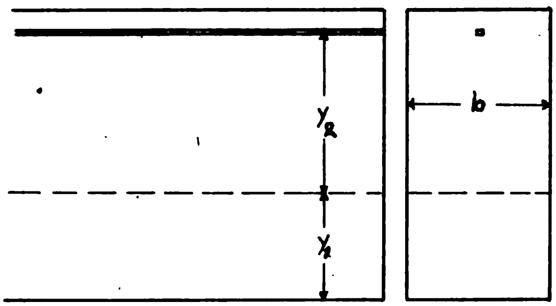

Fig. 33.

for moment of resistance the equation

$$M_0 = \frac{F a^2 b}{d}\left(y_2 + \frac{2 y_1}{3}\right) + \frac{8}{10} f_t b y_2 \left(\frac{y_2}{2} + \frac{2 y_1}{3}\right)$$

In these equations,

F = elastic limit of steel in pounds per square inch,

f_c = compression strength of concrete per square inch,

f_t = tensional strength of concrete per square inch,

b = breadth of section, in inches,

a^2 = area of one bar,

d = spacing of bars in inches,

$\frac{a^2 b}{d}$ = total quantity of metal in width (b),

M_0 = ultimate moment of resistance in inch pounds,

E_c and E_s = moduli of elasticity of concrete and steel, respectively,

y_1 = distance from compression face to neutral axis,

y_2 = distance from neutral axis to bar.

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*Discussion of Concrete-Steel Beams, Trans. A.S.C.E., Vol. 46.

As concrete is cracked long before failure, the last term in equation for moment of resistance will be omitted.

Thatcher's Formulas.

Thatcher's formulas* give for neutral axis the equation

$$x = \sqrt{\left(2A \cdot \frac{E_s}{E_c}\right) d + A\frac{E_s}{E_c}} - A\frac{E_s}{E_c}$$

for bending moment

$$M_o = \frac{S}{36}\left[\frac{E_c}{E_s} \cdot \frac{x^3}{y} + 3Ay\right]$$

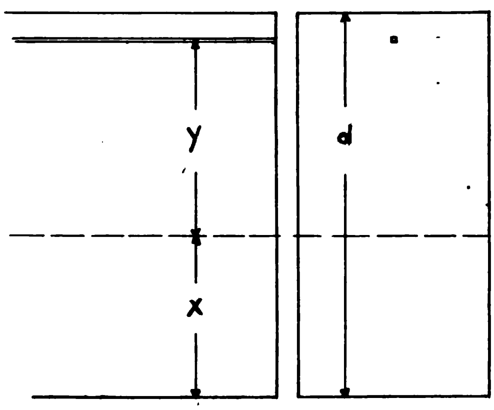

Fig. 34.

In these equations

x = depth from compression side to neutral axis,

y = depth from neutral axis to center of steel bar,

A = area of steel in tension, for one inch width of beam,

E_s and E_c are moduli of elasticity of steel and concrete, respectively,

M_o = moment of resistance.

Wason's Formula.

Computations of the breaking load were made also with a formula deduced by L. Wason, similar to the formula used

*Transactions of the Association of Civil Engineers of Cornell University, Vol. X.

by the Ransome company. In this formula several arbitrary assumptions are made, which are not quite the same as are made in the other formulas. The Neutral axis is assumed to be half way between the compression face of the beam and the steel rod. The center of pressure of the cross-section under compression stress is equal to two-thirds the height from the compression face to the neutral axis. Then the distance from the neutral axis to the center of the bar is $\frac{5}{6}$ D, where

D = depth in inches from compression face to center of bar;

l = span in inches;

W = total load in pounds;

f = total fiber stress in the steel.

The moment of resistance, assuming the concrete to take all the compression, and the steel all the tension, equals

$$M_o = \frac{5}{6} Df$$

The bending moment on a simple beam bearing two concentrated loads equal and symmetrical with respect to the middle of the beam is

$$M = Pa$$

$$\text{Then } P = \frac{5Df}{6a}$$

Table No. 3

No	Yield Point					Failure								
	Actual	J.B. Johnson		W.K. Hatt		Actual	A.L. Johnson		W.K Hatt		E Thacher		Wason	
	Load	Y_2	Load	hx	Load	Load	Y_2	Load	hx	Load	x	Load	½ D	Load
Group 1														
2	1250					3000								
4	1250	3.31	1675	3.2.3	1490	2000	3.48	3250	1.05	3950	2.69	2850	2.5	3500
Av'ge														
Group II														
36	1600					5250								
35	1200	3.49"	1920	3.38	2050	4000	3.48	5800	1.88	6450	2.87	10400	2.5	6250
Av'ge						4370								
Group III														
22	1600					6500								
19	1700	3.49	1920	3.38	2050	4200	3.48	5800	1.88	6450	2.87	10400	2.5	6250
Av'ge						5560								
Group IV														
34	2000					5800								
31	1600	3.49	1920	3.38	2050	5000	3.48	5800	1.88	6450	2.87	6600	2.5	6250
Av'ge.						5260								
Group V														
24	2000					5500								
11	1500	3.49	1920	3.38	2050	4400	3.48	5800	1.88	6450	2.87	10600	2.5	6250
Av'ge						4930								
Group VI														
16	2000					5600								
15	1800	3.49	1920	3.38	2050	4900	3.48	5800	1.88	6450	2.87	10400	2.5	6250
Av'ge						5225								
Group VII														
5	1200					6200								
10	1800	3.49	1920	3.38	2050	4200	3.48	5800	1.88	6450	2.87	10600	2.5	6250
Av'ge						5360								

Table No 3.

No	Yield Point					Failure								
	Actual Load	J.B. Johson		W.K. Hatt		Actual Load	A.L. Johnson		W.K. Hatt		E. Thacher		L. Wason	
		y_2	Load	hx	Load		y_1	Load	hx	Load	x	Load	½D	Load
Group	VIII													
25	2400					6000								
28	2400	3.49	1920	3.38	2050	5800	3.48	5800	1.88	6450	2.87	6600	2.5	6250
Av'ge						5900								
Group	IX													
57	900					6420								
62	1000	3.49	1920	3.38	2050	4600	3.48	5800	1.88	6450	2.87	10600	2.5	6250
Av'ge						5400								
Group	X													
45	2200					6300								
46	2000	3.39	2340	3.32	2290	6500	348	580	1.40	6700	2.62	6600	2.5	6250
Group	XI													
48	2000					5920								
49	2000	3.39	2340	3.32	2290	6000	3.48	5800	1.40	6700	2.62	4200	2.5	6250
Group	XII													
51	1800					6000								
52	2000	3.39	2340	3.32	2290	6600	3.48	5800	1.40	6700	2.62	6450	2.5	6250

Formula for Computing Elongation from Measured Deflection.

The relation between deflection and elongation of extreme fiber at yield point was deduced as follows.

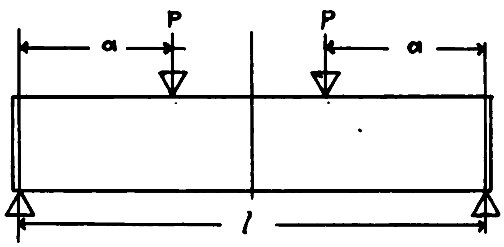

Fig. 35.

Maximum deflection $= \Delta$

S = elongation of extreme fiber per inch;

P = concentrated load;

A = distance load to support;

1 = span,

c = distance neutral axis to extreme fiber.

From the equation of the elastic curve

$$\Delta = \frac{Pa}{6EI}\left(\frac{3}{4}l^2 - a^2\right) = \frac{Pa}{24EI}\left(3l^2 - 4a^2\right) \qquad (1)$$

From equation of moment of resistance,

$$f = \frac{Mc}{I} = \frac{Pac}{I} \qquad (2)$$

Solving for P in (1) and (2) and equating

$$P = \frac{24EI\Delta}{a(3l^2 - 4a^2)} = \frac{fI}{ac} = \frac{SEI}{ac}. \qquad (3)$$

since $E = \frac{f}{S}$, $f = SE$.

$$\therefore S = \frac{24c\Delta}{3l^2 - 4a^2}. \qquad (4)$$

This equation was used in determining the computed elongations given in Table 1.

Series No. 2.

Description of Beams and Manner of Reinforcement.

The beams in this series are three-months beams, 2-1/2" x 2-1/2" x 26". Two of them are unreinforced and the remainder are reinforced with three Bessemer steel wires, equally spaced and varying in size from 1/4 inch in diameter to 1/16 inch. These wires were placed two diameters from the surface of the beam in all but two beams where they were placed 1-1/2 diameters below the surface. The wires were cold drawn, varying in elastic limit from 65000 pounds per square inch for the 1/4 inch wire to 87000 pounds per square inch for the 3/32 inch wire. It was impossible to get the elastic limit of the 1/16 inch wire with the apparatus at hand, but judging from the variation in elastic limit of the larger sized wires, it must have been about 90000 pounds per square inch. The modulus of elasticity was about 28,000,000 in all cases. There were seven sizes of wire used, two beams being made with each size of wire. Six extra beams were made, two unreinforced, two with 1/4 inch wire placed 1-1/2 diameters below surface, and two with 5/32 inch wire. Thus there are ten sets of two beams each.

The objects of this series were: To determine the effect of varying percentages of metal, to get tests of beams in which the ratio of height to length was as 1 is to

PLATE X.

Table No 4. Data of Beams in Series No 2.

NO.	SIZE of RODS	YIELD - POINT					LOAD at Failure	TENSION TESTS		COM-PRESSION TESTS
		LOAD	DEFL'N.	ELONGATION						
				Micrometer	Extensometer	Computed		7 days	28 days	
47	-	330	0.008	0.00010	-	0.00016	340	280	495	5500
50		330	0.005	0.00010	-	0.00010	350	215	397	4600
53	1/4"	875	0.017	0.00026	0.00008	0.00020	1600	198	440	5100
54	"	750	0.015	0.00018	0.00005	0.00018	1520	198	440	5100
55	"	800	0.010	0.00032	0.00006	0.00016	1000	245	440	-
56	1/4"	700	0.011	0.00016	-	0.00018	1600	255	440	-
59	7/32"	800	0.013	0.00028	0.00010	0.00021	1700	265	415	
60	"	825	0.013	-	-	0.00021	1900	255	415	
63	3/16"	800	0.016	0.00018	-	0.00026	1200	250	430	5000
64	"	600	0.012	0.00014	0.00005	0.00020	900	250	430	5000
65	5/32"	350	0.010	0.00010	-	0.00020	350	-	400	5600
66	"	400	0.008	-	-	0.00015	500	-	400	5600
67	"	550	0.012	0.00008	0.00008	0.00020	1200	370	415	5360
68	5/32"	550	0.013	0.00025	0.00010	0.00024	1120	370	415	5360
69	1/8"	500	0.010	0.00025	-	0.00020	1060	-	480	6500
70	"	500	0.010	0.00020	0.00010	0.00020	1260	370	480	6500
71	3/32"	500	0.000	0.00035	0.00024	-	920	-	510	5200
72	"	375	0.009	0.00016	-	0.00016	800	-	480	5200
73	1/16"	275	0.006	0.00010	-	0.00016	500	-	480	4400
74	"	400	0.011	0.00010	-	0.00020	550	-	480	4400

Note:- No's 53,54,- wires $1\frac{1}{2}$ diam's below surface.. On all others reinforced, wires are 2 diam's. below surface.. No's 65 and 66-mortar too dry- Adhesion between steel and concrete poor..

Fig.-36.- Typical Deflection Curves for Beams of Series No. 2

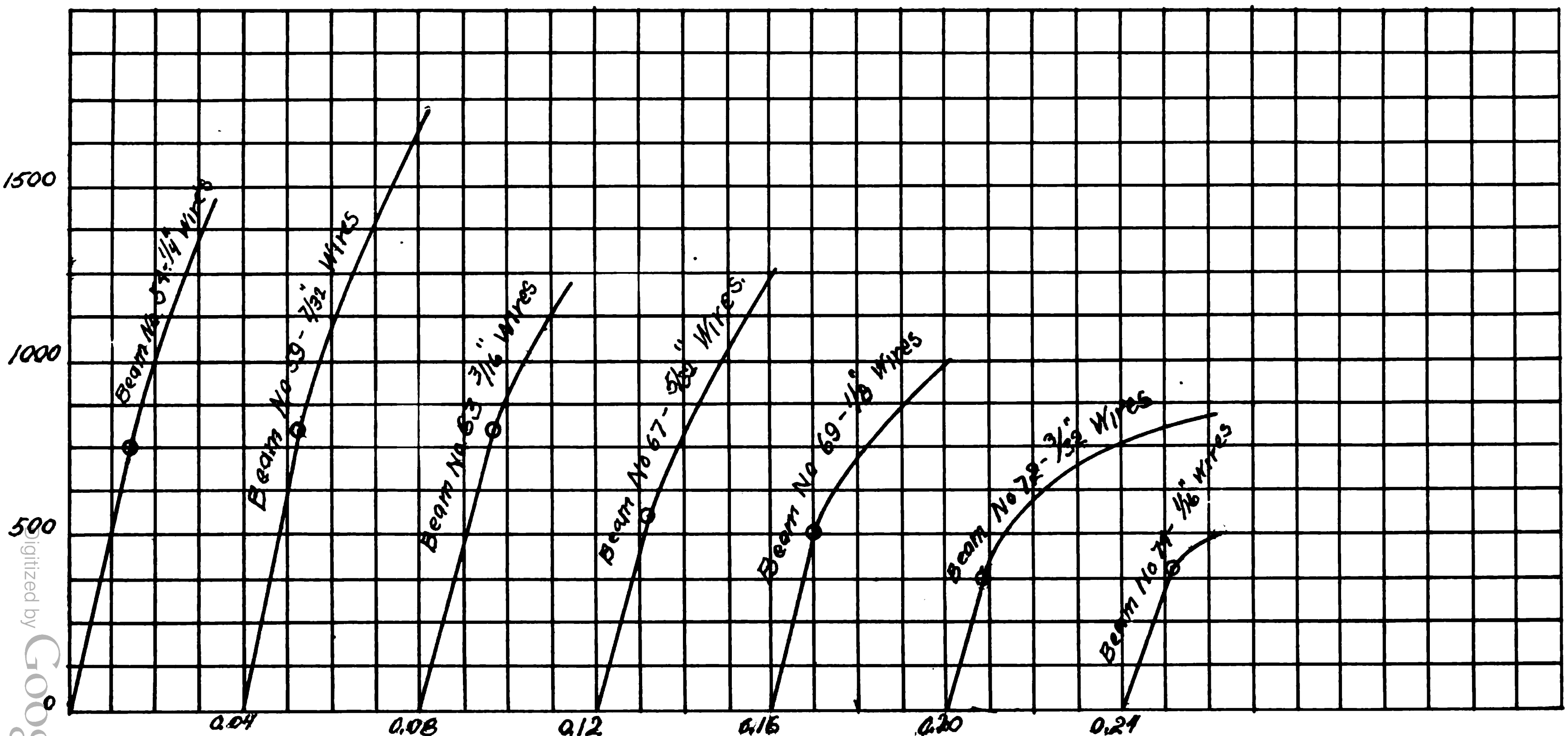

Fig. 37.- Elongation Curves for Beams of Series 2.

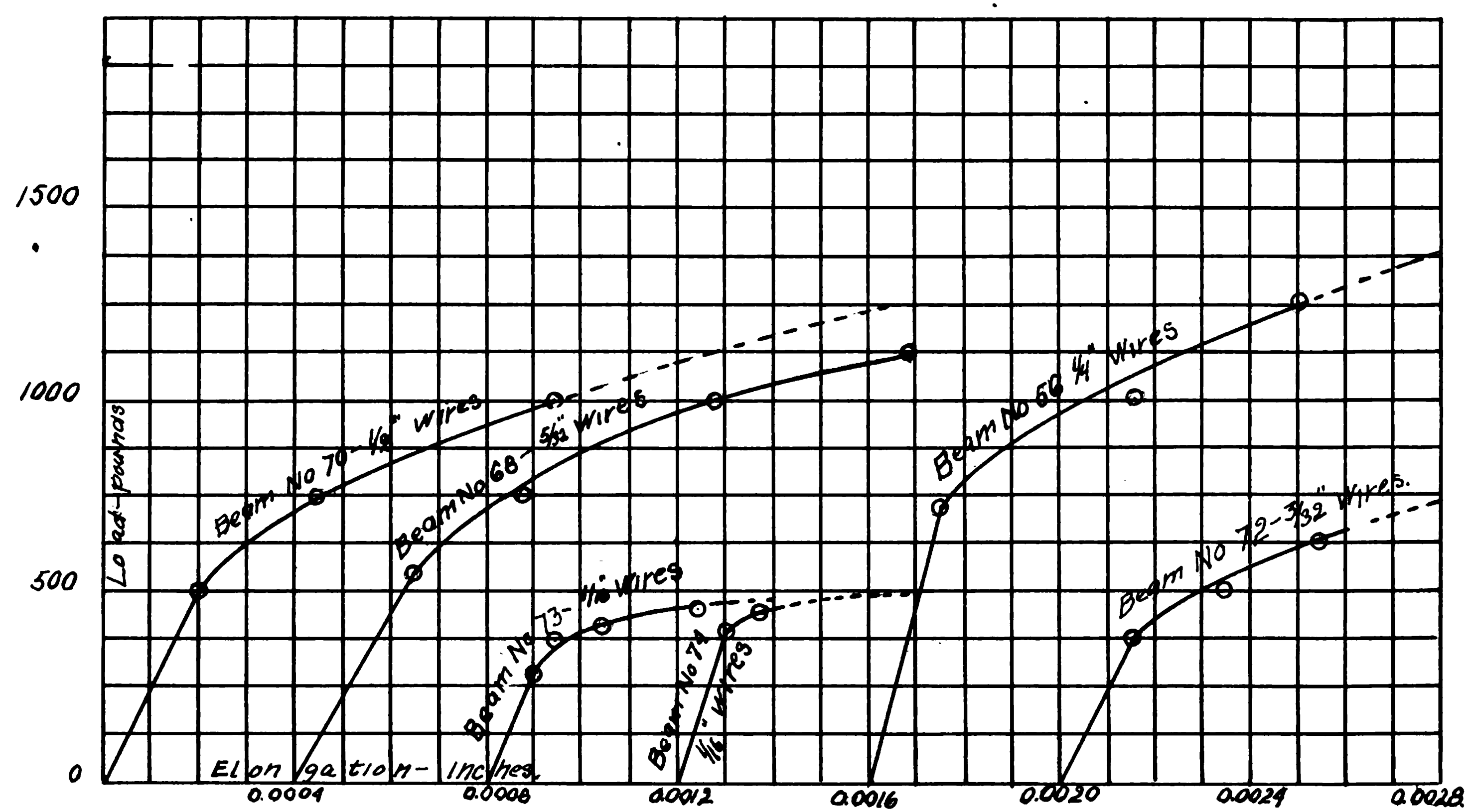

Fig. 38.— Showing a Curve plotted from Data obtained by M. Considere in his Test of Beam No 34.

10, to duplicate Considére's tests as nearly as possible and to determine the effect on elongation when the metal is finely distributed. As will be seen from the tables, the strength of the beams had a tendency to decrease as the percentage of metal decreased. That is, the yield point was lower, also the load of failure, in most cases.

A few of the typical deflection and elongation curves obtained from tests of this series have been plotted and can be compared with those of Series 1. See Figures 36 and 37.

Results as regards the Theory of Finely Divided Cracks.

Dean J. B. Johnson says,* "As shown, the steel cannot carry much load until the concrete cracks, but this does not open widely at any point because of its adhesion to the steel bar. It will open in a series of fine cracks." With reference to this it may be said that no such action was found in any of the tests of beams recorded in this paper. The cracks came at intervals varying from 1/2 to 6 inches in the large beams and from 1/2 to 3 inches in the small beams of this series. Plate IX illustrates how the cracks came on the beams.

Results as regards Other Objects of the Tests.

Since the metal in this series of beams was finely distributed, it is evident that the distribution of metal

does not affect elongation. The tables will show that the proportionate elongation, as measured, is practically the same at the yield point as that in the larger beams. Computed elongations, likewise, are practically the same.

The opinion was advanced that perhaps the ratio of height to length affected the elongation. For this reason the critical ratio of 1 to 10 was used in these beams. As was seen above, the elongations showed no appreciable difference.

Comparison of Considére's Beam No. 34 with Beam of this Series.

With regard to a direct comparison of the laws of this series and those tested by M. Considére, the load elongation curve has been plotted for beam No. 34, to which reference has been made previously. The following table gives bending moments and elongations obtained from the test of beam No. 34, as recorded by M. Considére.* These values have been transformed into units corresponding to those in which the beams of this series are recorded. Figure 38 shows the curve plotted from these loads in pounds, and elongations per inch.

*Influence des Armatures Metallique sur des Proprietés des Mortéres et Bétons, Le Génie Civil, Feb. 4, 1899, P. 215.

Table No. 5.

Moments de flexion Supportés par le prism	Allongements Constatés de Béton	Bending moments in foot pounds	Load in pounds $P = \frac{M}{a}$	Elongations in inches per inch
Kgm.	Mm.			
5.18	0.038	37.5	112.5	.0000385
11.48	0.092	80.7	242.1	.000093
19.88	0.186	144.0	432.0	.000188
30.38	0.424	220.0	660.0	.000430
40.18	0.775	296.0	880.0	.000785
49.28	1.050	358.0	1074.0	.001060
63.98	1.520	464.0	1392.0	.001540
78.68	1.980	570.0	1710.0	.002000

The size of wires used in Considére's beam No. 34 was about the same size as the 67 and 68 of this series. A comparison of Considére's curve with that of beam 68, Figure 37, shows that the yield point of his curve is somewhat lower. His elongation, at yield point, is 0.0002 inch while that of beam 68 is 0.00025 inch at the yield point. Thus it is seen that the elongation he obtained is not even as much as that of beam 68. The load at failure of his beam, computed for a system of loading similar to ours, is much higher than that of beam 68, which may be due to different metal and concrete in the two beams. A further examination of the curve shows that if the quoted elongation on the curve be compared to that at the yield point, it will be found to be ten times

as great. Thus it is easily seen how he obtained his ratio of 1 to 10 in the elongation of reinforced beams.

There is a distinct yield point shown on this curve, the same as in all the curves of reinforced beams in these two series, and there is no doubt in our minds that the cracks, which Considére found, occurred at this yield point. Thus, from this curve and from what M. Considére himself has said about this beam, it is evident that the width of the cracks is included in the measurement of his elongations. For this reason we feel justified in saying that while M. Considére's beam, elongated as to its extreme tension face ten times more than his unreinforced beam, the mortar on the tension side did not elongate any more than the mortar on the tension side of his unreinforced beam.

Summary.

From the tests recorded in this paper, the following conclusions may be drawn. The elongation of reinforced concrete under tensional stress in a beam is not more than that of unreinforced concrete under the same conditions, up to the point at which first crack appears. This holds true whether the metal reinforcement is finely distributed or not. When the proper constants are introduced, the formulas deduced by Mr. A. L. Johnson and Professor Hatt will apply very well in computing load at failure if the tensional strength

of the concrete is neglected. Professor Hatt's formula for computation of the various quantities at yield point applies in the tests above described. The method of computation given by Dean J. B. Johnson, for the load at yield point, agrees more closely with our results than any other which we have found. Mr. Edwin Thatcher's formula does not agree with the results of our tests, probably because it was not designed for use with reinforcement of such high ultimate strength.

With regard to the action of water on the strength of concrete after setting, it would seem that reinforced concrete immersed in water has a greater strength at yield point but that reinforced concrete, which remains dry after setting, has a greater ultimate strength. The elongation and deflection at the yield point are not perceptibly affected by water or absence of water after setting.

Beams which have been stored in water, will show water-marks where cracks have occurred during the test, provided the surface is sufficiently dry. The first of these water-marks appears at the yield point, which is indicated by a definite bend in the curves of elongation and deflection and following directly from this, the concrete cracks at the yield point.

The concrete in reinforced beams does not crack at finely distributed points over the surface under tensional stress, but the cracks come at relatively large intervals apart.

The reinforcement of the beams increased the strength at yield point about fifty percent.

While these conclusions are regarded as absolutely true as regards the beams which we have tested, the similarity of our results and those quoted from Messrs. Hatt and Considere lead us to believe that our conclusions are true for reinforced concrete beams, in general.

Conclusion.

In conclusion we wish to thank Mr. Hartman for his efficient assistance in planning and carrying out the tests and in working up the data obtained. We are well satisfied with the benefit derived by us from the work and consider it due in a great measure to the interest which he displayed, from first to last, in our work.

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